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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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CONSTRUCTION METHODS ON THE MOFFAT TUNNEL

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Synopsis

This paper describes in detail the interesting methods used in the excavation of the Moffat Tunnel in Colorado. This work is really two parallel headings constructed at the same time, one a single-track railroad tunnel and the other a pressure tunnel for water.

There is also included a brief statement of the location and purpose for which the tunnels are built, as well as some notes on the financing of the work by the Moffat Tunnel Commission representing the Moffat Tunnel Improvement District, the form of contract for the construction, and the workmen's bonus plan.

The Moffat Tunnel is the longest railway tunnel in America, and the methods used in its construction are believed to represent the best present-day practice.

This paper was written before the completion of the project and describes methods used only up to August 1, 1925, at which date the headings were approximately 62% completed.

Selection of Site

For many years the general location selected for the Moffat Tunnel has been considered the best place, in fact, the only practicable location in Northern Colorado for building a railroad through the Continental Divide. At this point the approaches on both east and west sides are most favorable.

At various times tunnels have been laid out at different elevations and of different lengths, the higher ones of course being shorter and having the merit only of relative cheapness. The Moffat Tunnel is at the lowest practicable elevation. Any further lowering would require a much longer tunnel and would not improve the alignment or grades of the railroad on either side of the Divide. The work is at an elevation of approximately 9 200 ft. and is 32 250 ft long.

Physical and General Conditions

The tunnel is to be used by the Denver and Salt Lake Railroad—usually known as the Moffat Road—which has been in operation for twenty years. The original builders always had in mind that ultimately a tunnel would

Note.—Written discussion on this paper will be closed in May, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

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be built at this location but were induced to locate a line "over the hill" as a temporary expedient. The gradient of the railroad from the East Portal to Denver, Colo., 50 miles east, is 2%, down-hill practically all the way. On the west side the grade from the West Portal to Tabernash, Colo., 9 miles beyond, is 2% down-hill and thence 1% down-hill to State Bridge, 60 miles farther on. As shown on Fig. 1, it is intended eventually to build a connection between the Moffat Road and the Denver and Rio Grande Western Railway, the so-called Dotsero cut-off. By developing the line between Tabernash and the West Portal (Fig. 2), it is feasible to build all the way from the Denver and Rio Grande Western Railway to the West Portal on a 1% ruling grade against the traffic. There is also a project on foot to build a connection between Salt Lake City, Utah, and Craig, Colo., the western terminus of the Moffat Railroad.

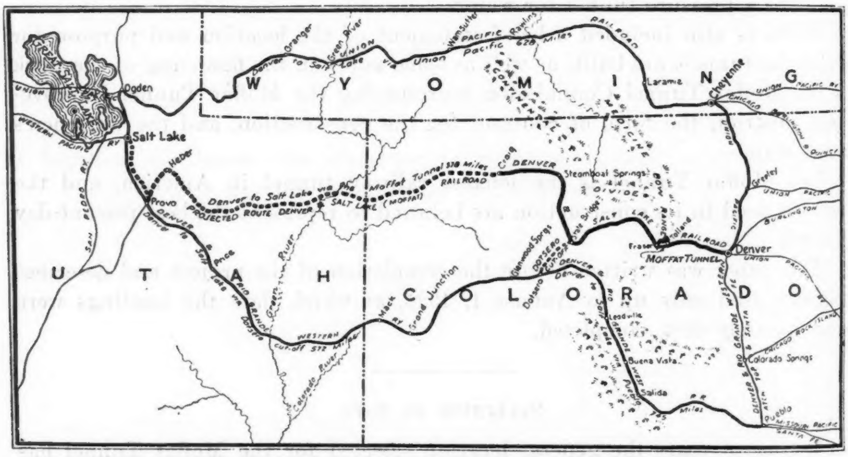

FIG. 1.—MOFFAT TUNNEL AND ITS RELATION TO WESTERN RAILROADS.

The line "over the hill", however, is built on a 4% grade from a short distance above the tunnel elevation to the summit at Corona, Colo., an elevation of 11 660 ft. Snow, ice, and wind conditions are not serious matters up as far as the elevation of the tunnel, but above this point the conditions become much worse up to the top of the range. The significance of a 4% grade at such altitudes is perhaps not fully realized without taking into account the difficulties added by the weather conditions. These are perhaps at their worst in March and April, the first warm days of spring causing some melting of snow which in the form of water finds a natural channel following the rails. At nightfall, or on entering a snow-shed, this flowing water freezes, covering the rails with ice, which no snow-plow can remove; in addition to which there are snow drifts 20 ft. deep caused by winds as high as 90 miles per hour. A trip over this section in winter is truly an adventure, and as to the cost of operating "over the hill", it is stated that 41% of the entire operating expenses of the railroad are spent at this point on the line. The need of the Moffat Tunnel is thus obvious.

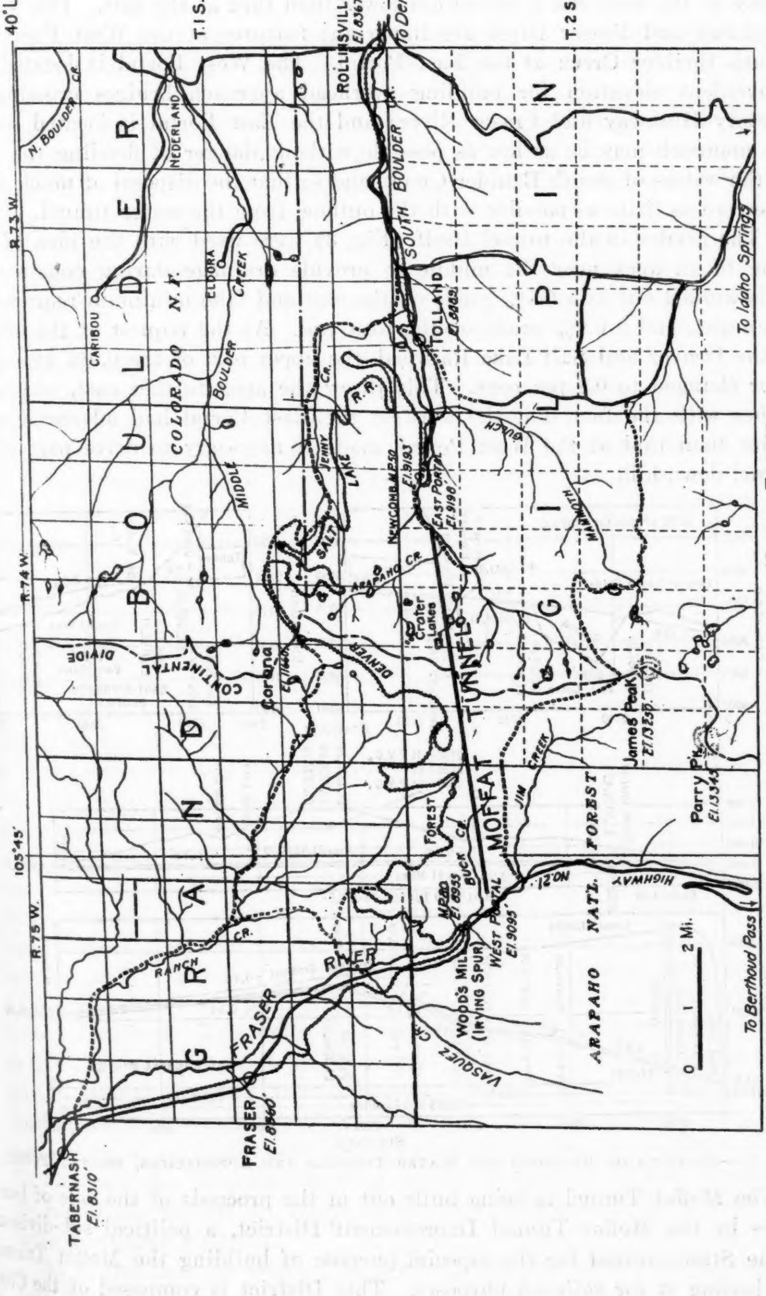

FIG. 2.—LOCALITY MAP, MOFFAT TUNNEL.

Speaking now more particularly of the especial considerations governing the exact layout of the tunnel, it will be noted from Fig. 2 that the topography at the west end is somewhat lower than that at the east. The Victory Highway and Fraser River are important features at the West Portal and South Boulder Creek at the East Portal. The West Portal is located at a convenient elevation for building overhead approach bridges crossing the Victory Highway and Fraser River; and the East Portal is located so that the approach may be as low as possible without danger of flooding the track by the waters of South Boulder Creek, and so that the disposal of muck would interfere as little as possible with the outflow from the water tunnel.

The grades in the tunnel itself (Fig. 3) were fixed with the idea of running to an apex near the middle to provide drainage during construction. This worked out as a 0.3% grade on the east end (the minimum required for drainage), and a 0.9% grade on the west end. At the request of the officials of the Denver and Salt Lake Railroad, the upper part of the 0.9% grade was later changed to 0.8 per cent. This threw the apex farther east, which, together with the fact that the work at the East Portal had advanced much faster than that at the West Portal, made it necessary to drive part of the tunnel down-hill.

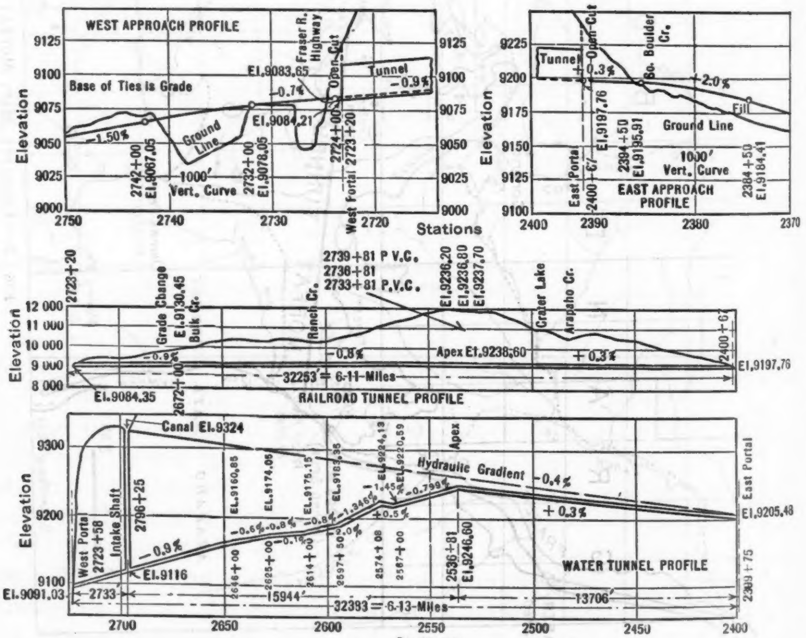

FIG. 3.—PROFILES OF RAILROAD AND WATER TUNNELS AND APPROACHES, MOFFAT TUNNEL.

The Moffat Tunnel is being built out of the proceeds of the sale of bond issues by the Moffat Tunnel Improvement District, a political sub-division of the State created for the especial purpose of building the Moffat Tunnel and leasing it for railroad purposes. This District is composed of the City of Denver and the whole northwestern part of the State. It was required

by the law creating the District that provision should be made in building the tunnel for the conveyance of water from the western to the eastern slope of the Divide as a prospective domestic water supply for the City of Denver to the amount of 100 000 acre-ft. per year. It was decided by the Board of Consulting Engineers for the Moffat Tunnel Commission that this object could be best obtained by constructing a separate parallel and smaller tunnel for the conveyance of water only. Such a plan would incidentally enable both tunnels to be advanced simultaneously by the so-called pioneer, or better the twin-heading method, a plan similar to those used on the Simplon and Rogers Pass Tunnels. Such a plan is very desirable for purposes of ventilation, if for no other reason.

The water tunnel was located 75 ft. south of the center line of the railroad tunnel (Fig. 3). In grade the center line of the water tunnel follows approximately the center line of the railroad tunnel. This design requires the west end of the water tunnel to be under pressure. The hydraulic grade originally designed to be 0.3% was later changed to 0.4 per cent. The necessary pressure at the west end will be obtained by constructing a shaft about 265 ft. deep (Fig. 3) to connect with collection ditches on the western slope. The shaft is located about 2 500 ft. in from the end. The water tunnel will be closed by a concrete plug west of the shaft, thus abandoning the west end of the tunnel which will have served no purpose except to facilitate the construction of the remainder of the water tunnel and of the whole railroad tunnel.

Geology of the Region and Its Effect

Briefly, the whole region is part of the great uplift of the Continental Divide. Practically all the rock in the immediate neighborhood of the tunnel is biotite granite gneiss. There are some pegmatite intrusions, particularly one directly at the West Portal, which determined the exact location of the portal as it was the only rock outcrop in this vicinity. In a long stretch of tunnel from the West Portal eastward the rock has been profoundly disturbed by geologic action probably incident to the mountain-making movements.

The pegmatite which extended in about 500 ft. from the West Portal turned out to be somewhat weathered. Thence, for 1 100 ft. the material almost resembled the output of a crusher, dust and all, containing enough water to form a cohering mass. There was some semblance of bedding planes, however, and practically all the fragments showed slicken-sided faces. For a further distance of 6 400 ft. the formation is different in that the faulting is more localized. The material in some of these faults was dry and crushed so fine that when exposed by blasting it would run into the tunnel like dry sand. Several of these faults, however, carried water in addition to the crushed material which complicated matters. As to soundness, however, the rock condition averaged considerably better the farther the headings advanced. To illustrate the irregular faulting in this formation, it was observed that the conditions at opposite points in the two tunnels only 75 ft. distant were invariably quite different.

As to the limit to which this faulted zone extended, it is interesting to note that east of the point where the abrupt western slope of the range near

the summit intersects the tunnel grade the rock formation is relatively sound, which would indicate that the strata forming this slope extend to the tunnel grade. The eastward extension of these strata, all biotite granite gneiss, is terminated by a vertical fault at the crest of the range forming an eastward facing escarpment several hundred feet high. The escarpment is not continuous in this vicinity, however, being broken at intervals by several narrow granite formations extending eastward for various distances from several hundred feet to ½ mile. The eastern slope of these granite formations is gradual enough to form a path for access to the top of the range, which at the tunnel line is 12 000 ft. in altitude.

The eastern slope of the range, as may be seen from Fig. 3, is much more precipitous than the western. An interesting feature of the topography of the eastern slope is the so-called Crater lakes (Fig. 2) formed by glacier action, the water being dammed up by moraines of glacial till. It was rumored that some of these Crater lakes were bottomless which would mean water all the way down to the tunnel level. In reality all these lakes appear to be shallow. In one case, however, there was doubtless a connection between one of the lakes and the tunnel by means of a faulted zone. This was demonstrated in the latter part of February, 1925, when in blasting a round in the water tunnel at the East Portal a sudden flow of water was encountered, amounting approximately to 1 800 gal. per min. This water flowed down the ditch alongside the track in the water tunnel and was diverted at one of the cross-cuts to the railroad tunnel.

Unfortunately, however, the large quantity of silt carried by this water filled up the ditches, as a result of which the track in the water tunnel was flooded. As any flows previously met were trivial in quantity, the Crater lakes were suspected as being the source of this water. One of these lakes was directly above the leak, at a level about 1 400 ft. above the tunnel. Although it was during the middle of winter, men were sent up to investigate conditions but found the lake covered by several feet of ice. Digging a hole in the ice at a convenient point near the middle of the lake, which was 10 acres in extent and about 10 ft. deep, they dumped in a package of chloride of lime on the remote chance that some might be carried into the tunnel by way of the "leak". Strange to say tests of water two hours later in the tunnel showed the presence of the chloride of lime. After a few days the flow rapidly diminished to less than 100 gal. per min. This apparently was due to the silting up of the fissure, as a subsequent inspection of the lake showed no material lowering of the water level, and a further test with chloride of lime showed that the connection still existed between the Crater lake and the tunnel, the chemical coming through in 20 min.

At the East Portal the rock was granite gneiss and schist, and the bedding nearly vertical, the strike of the beds being at an angle of about 15° with the axis of the tunnel. This rock has been faulted somewhat, the faults following the bedding approximately. It is curious that at the fault where the flow of water was encountered in the water tunnel practically none was found in the railroad tunnel heading only 75 ft. away.

No minerals have been found in the excavations. The problem of high rock temperatures as in some of the Alpine tunnels, notably the Simplon,

seems to be non-existent at the Moffat Tunnel. Therefore, not much attention has been paid to temperature measurements, but it is interesting to note that near the East Portal the rock temperature as measured in drilled holes was about 60° Fahr. 1 000 ft. in from the East Portal and 68° at a point 8 000 ft. from the portal where the cover was 1 400 ft. At the West Portal there was little increase of temperature with the greater cover, which was probably due to the cooling effect of descending ground-waters.

Overlying the unsound rock section near the West Portal for about 2 500 ft. from the portal the surface of the ground averages 250 ft. above the tunnel level. Over the greater part of this area the depth to rock is not known, although attempts were made before the tunnel was started to determine it. One test pit was sunk by hand in the overlying glacial drift to a depth of 75 ft. Progress was then so slow and expensive that a well drill contractor was employed to start a hole in the bottom of the test pit with a well drill outfit of the percussion type. Curiously enough, within 1 ft. of the bottom of the test pit, rock was struck—hard red granite—and after drilling a hole for 10 ft. into the formation it was claimed that this was ledge rock. To test this theory another hole was started near-by drilling from the surface through the glacial drift. Practically no progress could be made, however, and the hole was soon abandoned. By this time the tunnel was being excavated from the portal and no further investigation by borings was made. Other test pits in the meantime had been abandoned on account of water. The uncertainty of the position of rock at the West Portal was the cause of moving the tunnel at the west end 400 ft. south from the previously accepted location. This lengthened the tunnel slightly but gave considerably more cover with the likelihood of a greater depth of rock overhead.

This change in alignment was adopted in August, 1923, at which time the final layout was approved. This fixed the grades, the location of the water tunnel 75 ft. to the south of the railroad tunnel, and the shifting of the East Portal 75 ft. north of the previously accepted location, which last change allowed more room on the south side of the canyon to take care of the waters of South Boulder Creek and the water from the water tunnel.

Form of Contract

On account of the hazardous nature of the work it was considered doubtful whether bids could be obtained from reputable contractors for constructing the tunnel on a unit price basis. Therefore, the law creating the Moffat Tunnel Improvement District and providing for the appointment of a Commission to go on with the work permitted the Commission to construct the tunnel with its own forces, or to make a contract for the construction on some other basis than the usual unit price method.

Specifications and plans were prepared, however, on the assumption of constructing the tunnel by contract on the unit price basis. Proposals were asked from reputable contractors on this basis, the bids to be submitted September 12, 1923.

However, no bids were submitted except one to which had been added qualifying conditions making it irregular. This bid was held temporarily for investigation and it was stated informally by the Moffat Tunnel Com-

mission that the work would have to be done in some other way than by the straight contract unit price method. On this announcement about eight more informal bids were submitted made up in eight different ways, all being variations of the straight percentage contract or "cost plus".

These proposals were not solicited by the Commission and created considerable confusion as to their relative merits so that it was decided by the Board of Consulting Engineers, in consultation with the Moffat Tunnel Commission, to prepare a proposed form of contract on the cost plus basis and to ask bids thereon from only four of those who had previously submitted informal bids.

The governing features of this form of contract were as follows: Plans and specifications as already prepared were to be closely followed with the single notable exception that the work involved in handling "free flowing mud or sand or the influx of water in such volumes or temperatures as to render the work impractical without special drainage or cooling devices", was to be considered as extra work to be paid for in addition to the regular items of the contract. Ordinarily a contractor is required to assume such risks and in transferring the entire risk to the Moffat Tunnel District it was expected that contractors would be more inclined to submit bids.

Otherwise the form of a contract devised by the Board of Consulting Engineers can be described as a "cost plus a maximum and minimum fee" contract. The maximum fee was to be paid to the contractor in the event of the cost of the work being less than a certain assumed total upset price and the minimum fee in the event of the cost of the work being more than the upset price. The form of contract also contained a profit-sharing clause, namely, in the event that the cost of the work was less than the upset price one-half the saving was to be paid to the contractor as additional compensation. There was also provided a time bonus and penalty on a daily basis for finishing the work before or later than a certain fixed date. All the extra work and all work not susceptible of classification under the regular items of the specifications were to be paid for at cost plus a percentage.

In asking for bids under this form of contract the total upset price was fixed at $5 250 000 on the assumption that the final estimated quantities would be the same as those proposed for the comparison of bids on the unit price basis. As the final quantities would no doubt differ from those in the preliminary estimate it was considered necessary to vary the upset price as the quantities varied. The bidders therefore were requested to submit a schedule of prices, which, multiplied by the estimated quantities, would amount to the upset price. The bidder was expected to make these prices reasonable in respect to one another, so that his proposition should not be rejected as being unbalanced.

Bonus, Fees, and Penalties

The time for the completion of the work was fixed at forty-six months from September 20, 1923. This would bring the contract date for finishing the work to July 20, 1927. The time bonus and penalty at the rate of $1 000 per day were to be calculated from this date.

It was further provided that all the work done and methods used were subject to the approval of the Chief Engineer of the Moffat Tunnel Commission, and that if the work were not prosecuted satisfactorily by the contractors the contract could be annulled by the Moffat Tunnel Commission.

For extra work the contractor was to be paid 5% of the cost thereof independently of all other provisions of the contract. All funds for every purpose were to be provided by the Moffat Tunnel Commission.

On these general terms the bidder was asked to state a maximum and minimum fee for which he would agree to supervise the work. F. C. Hitchcock, M. Am. Soc. C. E., and Mr. C. C. Tinkler jointly submitted a bid naming a minimum fee of $140 000 and made their maximum fee the same. This was lower than that of any of the other contestants and as their bid was satisfactory in other respects it was decided to award them the contract.

This form of contract has some novel features. In order to make these entirely clear several examples of total payments under various assumptions of cost and time required for completion are given.

1.—Presume that the final upset price is $5 200 000; that the actual cost including fee is $4 640 000; that the percentage work is $200 000; and that the contract is completed 100 days ahead of time. The contractors will receive:

One-half of savings in cost	$280 000
Fee (maximum)	140 000
Time bonus	100 000
5% of $200 000	10 000
	$530 000

2.—Presume the same final upset price, the cost as $5 500 000, the percentage work as previously, and the contract completed 100 days ahead of time. The contractors will receive:

No savings	
Fee (minimum)	$140 000
Time bonus	100 000
5% on $200 000	10 000
	$250 000

3.—Presume the same conditions as in Example 1, except that the work is finished 100 days behind time. The contractors will receive:

One-half of saving	$280 000
Fee (maximum)	140 000
5% on $200 000	10 000
	$430 000
Time penalty	100 000
	$330 000

Time penalties, however, cannot operate to reduce the payment to the contractor, not including percentage work, below the minimum fee of $140 000. The large time bonus or penalty, $1 000 per day, represented at the time the contract was let approximately the cost of carrying the investment. On account of an additional bond issue sold to provide for an unlooked for increase in the cost of the work and also to defray the cost of supplementary work not included in the original contract, the value of speed of completion later became worth considerably more than $1 000 per day. Under these circumstances obviously a policy of energetic and rapid prosecution of the work was the only course. It is probably true that this policy was also the most economical in other respects.

To this end the contractors who, on being awarded the contract, organized the corporation of Hitchcock and Tinkler, Inc., to take over the work, decided to install the most modern and complete plant and equipment in all departments, to build, equip, and maintain high-class camps, and to pay wages and salaries high enough to attract the best class of workmen.

Construction Begun by Commission

On account of the urgency in getting the work under way the Moffat Tunnel Commission several months before the actual letting of the contract for the tunnel construction, had contracted with the Colorado Power Company for the erection of a transmission line from its power-house on Boulder Creek to the East Portal of the tunnel and thence over the Divide to the West Portal, and also for the furnishing by the same company at both portals of electric power at 44 000-volt, three-phase, 60-cycle, alternating current. The Commission had also authorized the beginning of camp construction and opening up the portals of the tunnels by hand, which work had been started as early as the middle of July, 1923. Some of the heavy machinery had also been ordered to insure delivery before winter.

General Plan of Operations at the East Portal

The details of the water and railroad tunnels, both unlined, are shown in Figs. 4 and 5. A view inside the water tunnel is shown in Fig. 6. The work of preparation of the water tunnel for the actual carrying of water was not included in this contract.

It had been recommended by the Board of Consulting Engineers that the grade of the water tunnel should be at mid-height of the railroad tunnel, on the assumption that the water tunnel would always be in advance of the railroad tunnel, and that cross-cuts could be driven at convenient intervals from the water tunnel to the center line of the railroad tunnel from which point another heading was to be driven, called the main heading of the railroad tunnel, and located on its center line both for line and grade (Fig. 5). This main heading of the railroad tunnel was to be of the same dimensions as the water tunnel, was to be driven both ways from each cross-cut, and all operations in the main headings were to be carried out by obtaining access through the cross-cuts (Fig. 4). At a convenient distance back of the face of the main headings ring-drilling operations were to be carried on for the enlargement of the main heading to a full-sized railroad tunnel.

Driving Headings, East Portal

Some doubt was expressed that this was really the best way. It was not considered a proved fact that the small water tunnel could really be driven faster than the railroad tunnel, for at this time hand-mucking operation in the water tunnel was the only method proven to be entirely feasible. Hand-mucking, as a matter of fact, was the only method used for several months, and the writer finds in his notes a statement to the effect that an average progress of 15 ft. per day was probably all that could be expected under the circumstances. However, at this time there had been developed in the lead mines of the Flat River, Missouri, District, the so-called Conway loader.

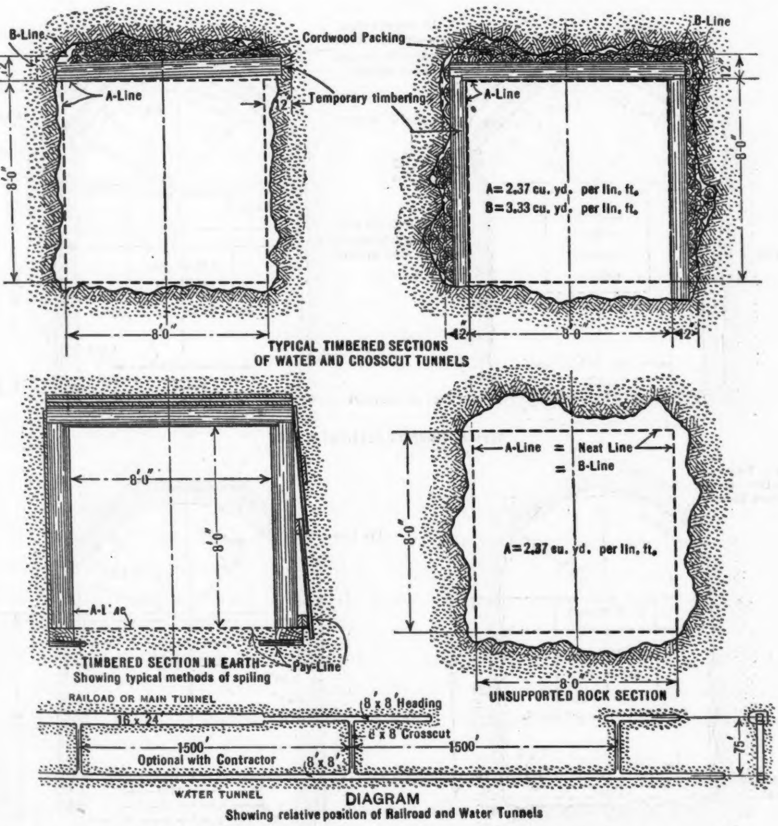

Fig. 4.—Water Tunnel Sections, Moffat Tunnel.

After investigation several of these loaders were purchased and proved to be a great improvement over hand methods of mucking. By the use of these machines an average progress of 24 ft. per day could readily be obtained. This result fully determined that the water tunnel could be driven faster than the railroad tunnel. Nor was this the only good feature connected with the use of the loaders. It was soon evident that a mucking crew could load

out all the broken rock from a heading in about the same time that a drill crew operating four drifter drills could set up, drill, and blast a standard round, nominally 8 ft. long. This led immediately to the adoption of the so-called "alternating method" of excavation wherein the main heading was driven only one way from a cross-cut, and the drilling and mucking crews alternated between the water tunnel and the single main heading. In this manner each crew had the field to itself for a period of approximately four hours, and great economies in labor were possible because none of the men in one crew found it necessary to wait on the other operation. Then, too, a spirit of competition was introduced between the two crews as to which could finish first.

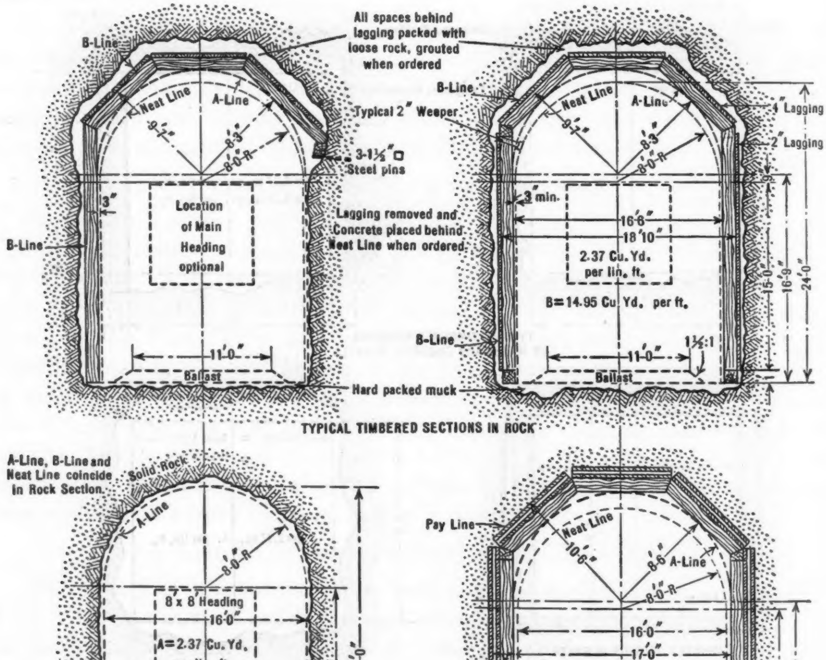

FIG. 5.—RAILROAD TUNNEL SECTIONS, MOFFAT TUNNEL.

Where each heading is driven independently both crews are in the heading at the same time and there is more or less interference. To minimize the interference the method of drilling is entirely different—only two drills are used, working from a single horizontal bar instead of four drills from two

columns; and a different layout of the drill holes is necessary for the reason that to save time as few holes as possible are drilled in the lower half of the tunnel.

After shooting, the heading is full of muck about two-thirds of the way to the roof. The drill foreman assigns one of his men to scale the roof and the remainder of the drill gang level off the top of the muck pile so that a horizontal bar may be set up about 3 ft. from the roof. On this bar two drills are mounted and drilling commenced immediately. In the meantime the mucking crew starts work and loads out the broken rock until the men get close to the bar when it is necessary to stop so as not to interfere with the drillers. The drillers continue drilling from the upper position of the bar until all the holes accessible from this position are finished. A horizontal V-cut or pyramid cut at the bottom of the heading is used.

The drill crew then removes its equipment and the mucking crew resumes work, cleaning up the heading. The drill crew then sets up its bar at what is called the lower position and drills four or more holes at the bottom of the heading, thus completing its work.

In the alternating method, with four drills working from columns in a heading free from muck, about 26 holes are drilled, making a regular vertical V-cut. The adoption of this method, giving each drill crew a clean heading in which to start work immediately, provided an ideal condition for using a drill carriage on which all the equipment could be mounted. With the older method all the equipment was moved by hand. Fig. 7 shows how the drill carriage operates. The horizontal bar, when in operating position, is jacked against the walls of the tunnel. The bar carries four arms, on each of which a drill is mounted. These arms are long enough for the drills to be placed in any position in the heading. When drilling is finished, the bar is turned on a vertical axis mounted on the end of a long cantilever extending forward from a small truck. In this turned position the drill equipment will clear the walls of the tunnel. For transportation to and from the heading, the drill equipment is drawn back by a sprocket chain and sheaves to a position immediately over the truck.

With such a mounting the bar and column arms are made much heavier than in the case of equipment intended to be worked by hand. This diminishes vibration and increases drilling speed. An automatic oiler and drill manifold are mounted on this same truck.

By the use of this carriage the average elapsed time from the arrival of the drill crew in the heading until the air was turned on the drills was 15 min. as against 40 min. in the case of hand-operated equipment.

The Conway loader (Fig. 8) is probably better adapted on the whole to the particular problem of loading muck in these headings than any other machine. In considering the choice of a shoveling machine the speed with which a car can be loaded is perhaps the least important qualification. The method of placing empty cars in position to be loaded is perhaps the most important. In the operation at the Moffat Tunnel a muck car with a capacity of 50 cu. ft. could be loaded under favorable circumstances in less than 1½

min., while it took about twice as long to remove the full car and place an empty one in position. This situation could be improved somewhat by using cars of larger capacity, which, in turn, since the car is attached to the loader while being loaded, would probably require that the back and forth motion of the Conway loader on the track be geared lower to diminish the strains incident to the rapid reversal of direction. All the cars were equipped with spring couplings.

The Conway loader is particularly good at loading scattered or so-called "fly" rock, which is really quite important. It is a rugged simple machine. Among its disadvantages are its limited range of operation on either side of the track and its ability to load only on a practically straight track.

Transportation to Headings

The transportation equipment used in the headings included storage battery locomotives, having a 3-ton chassis with Edison batteries, and 4-ton General Electric trolley locomotives. The muck cars were of 50 ft. capacity, having steel bodies dumping to one side only with an angle of dump of 37° and roller-bearing wheels The track was of 2-ft. gauge and the rail 40 lb. to the yard. Special flat cars were used for other purposes, particularly for transporting drill steel and men. These locomotives also served the ring drilling and other operations in the main heading but were used mainly only as gathering locomotives. All the long-haul work was done by 4-ton trolley locomotives operating in the water tunnel from the portal to the last cross-cut, at which point switching arrangements were such that transfers were readily made. All the trolley locomotives operated on 250-volt direct current.

Power for all operations in the tunnel was brought in on a 2 300-volt, three-wire, lead-covered cable. This voltage was used directly to operate 65-kw. motor generator sets generating 250-volt direct current for the trolley service and also for the Conway loaders and for low pressure fans forming part of the ventilating system. It was also used directly to operate 100-h.p. motors direct-connected to pressure blowers forming part of the ventilating system. For lighting purposes the 2 300-volt current was transformed to low-voltage alternating current. The use of high-voltage current in this way in the West is very general and quite successful.

Auxiliary Heading Method

An auxiliary tunnel to facilitate excavation has been used on a larger scale on two other tunnels—the Simplon Tunnel and Rogers Pass Tunnel—while in an incidental way it has been used no doubt in many other cases.

In the Simplon Tunnel the auxiliary heading, which was later to be enlarged to a single-track railroad tunnel, was at the grade of the bottom of the main tunnel, which was driven by the bottom-heading method, and the functions of the auxiliary heading were entirely devoted to ventilation and transportation. In view of the tremendous difficulties encountered, due to excessive heat and hot water, its use as such was indispensable. No attempt was made to keep the face of the auxiliary heading at a point opposite the main heading.

FIG. 6.—VIEW OF WATER TUNNEL.

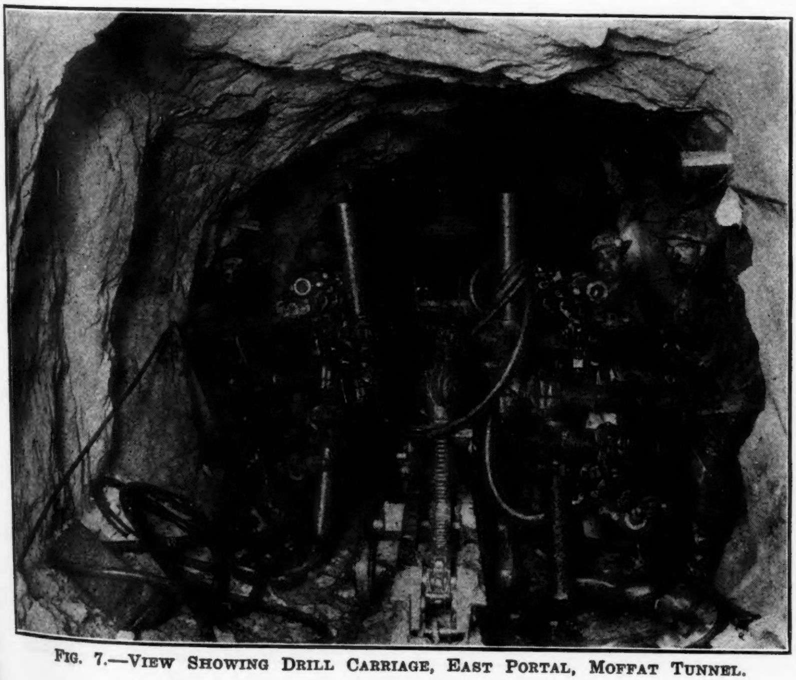

FIG. 7.—VIEW SHOWING DRILL CARRIAGE, EAST PORTAL, MOFFAT TUNNEL.

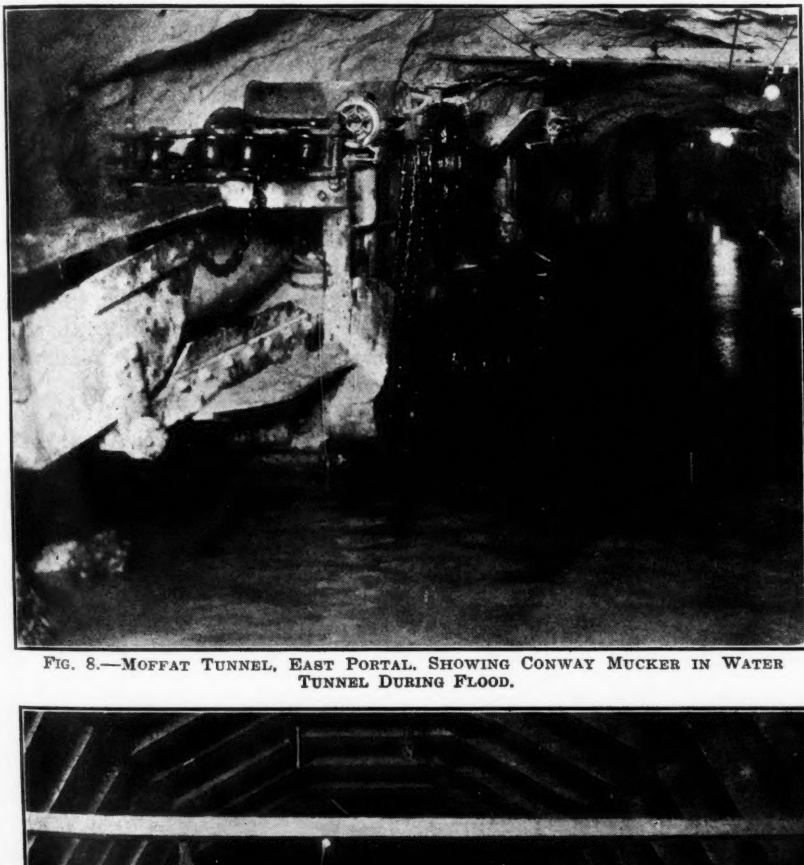

FIG. 8.—MOFFAT TUNNEL, EAST PORTAL. SHOWING CONWAY MUCKER IN WATER TUNNEL DURING FLOOD.

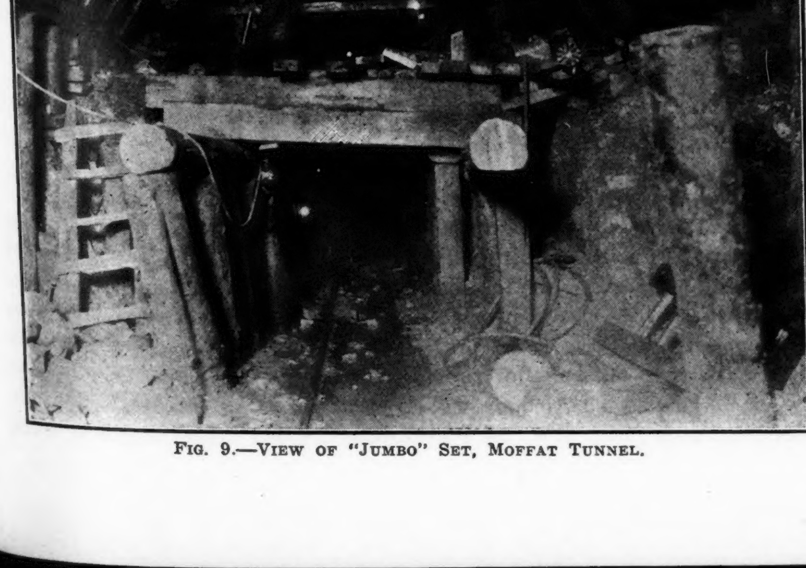

FIG. 9.—VIEW OF "JUMBO" SET, MOFFAT TUNNEL.

In the Rogers Pass Tunnel the purposes of the pioneer tunnel were the same as in the Moffat Tunnel—to facilitate transportation and ventilation and at the same time to afford a means of access to the ring drilling operations (which were first used on the Rogers Pass Tunnel). No mucking machines were used, however, all mucking being by hand, and the headings were driven independently of each other.

In driving the Moffat Tunnel the auxiliary heading proved particularly useful in avoiding inevitable delays, due to encountering areas of unsound rock usually crossing the tunnel at a small angle. If the railroad tunnel had been excavated without the aid of the water tunnel, these unsound areas would have caused great delay and expense due to the time necessary to place the timber. These areas were found in no less than fifteen different places, requiring timbering in lengths varying from a few feet to 500 ft.

Handling Unsound Rock

All this unsound rock was readily excavated and timbered far in advance of the regular enlargement operations. Access to the work was obtained through one of the cross-cuts, thus interfering in no way with the ring drilling or steam shovel work of the regular enlargement operations.

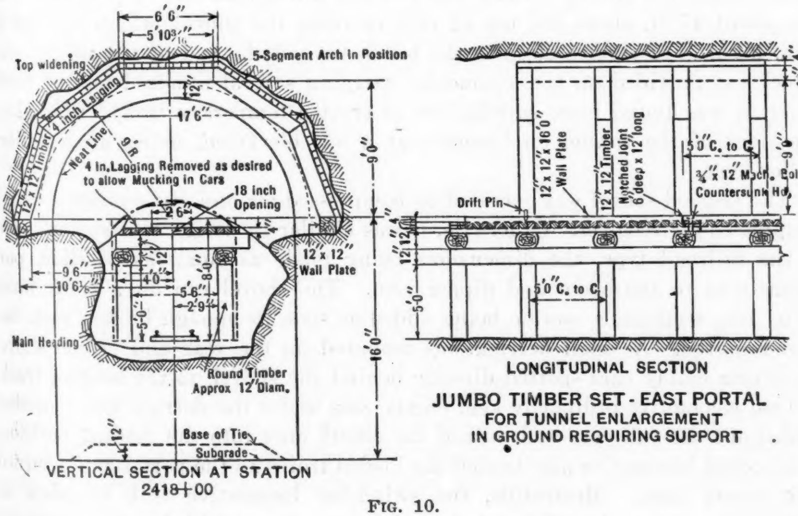

Fig. 10.

The method of excavating and timbering these areas was to build in the main heading throughout the unsound area a so-called "jumbo" (Figs. 9 and 10), which was merely a succession of heavy timber square-sets covered on top with heavy lagging. Under the protection of these timbers, transportation could be carried on without interference. The rock overhead was then stoped down and loaded by trapping methods into muck cars standing under the protection of the "jumbo". Timbering closely followed the excavation. Five-piece segmental sets of 12 by 12-in. timber supported on 12 by 12-in. wall-plates were used here. The wall-plates were set 19 ft. apart on a rock bench.

This distance is 1 ft. more than standard—to facilitate setting the posts under the wall-plates in the subsequent operations. Wall-plates were set about level with the top of the main center heading. After all the over-head timber was placed and all broken rock taken out, down to the level of the top of the main heading, the "jumbo" was removed and the material at the sides then excavated and loaded into cars by hand. This excavation was carried down only to the level of the bottom of the main heading, leaving an 8-ft. bench of rock which was excavated as part of the ring-drilling and excavation operation.

Enlargement Operations, East Portal

The standard method of enlarging the tunnel to full size at the East Portal was by the ring-drilling and shooting method. The broken rock was loaded into side-dump cars of nominal 4-yd. capacity, by a special railroad-type, Osgood, compressed-air shovel with a $\frac{3}{4}$-yd. dipper. Both cars and shovel ran on tracks of 3-ft. gauge. The dimensions of the shovel and cars were such that in a tunnel 16 ft. wide there was just clearance enough for the shovel to load a car alongside on a parallel track.

In the railroad tunnel 8-ton trolley locomotives were used. As in the water tunnel the trolley voltage was 250-volt direct current. The trolley wire was about 17 ft. above the top of rail, clearing the shovel, which was 16 ft. 7 in. high. For service close to the bench beyond the end of the trolley wire a reel was provided on the locomotive carrying several hundred feet of cable. Later, it was found more satisfactory to erect a temporary trolley wire along the sides of the tunnel on brackets at a low elevation, using an auxiliary trolley pole.

The Osgood shovel was operated by compressed air and was equipped with a compressed-air receiver. Otherwise, it was similar in design to a steam shovel of the railroad type, the dimensions being made as small as possible, particularly as to the boom and dipper arm. The shovel ran on a track made up in 5-ft. sections, a section being added as soon as enough broken rock had been removed. A small derrick was mounted on the rear end of the shovel for lifting empty cars spotted directly behind the shovel, to the loading track.

One locomotive ordinarily kept empty cars under the derrick while another hauled out the full cars and pushed the empty ones into the loading position. This second locomotive also hauled the loaded trains to the portal and returned with empty ones. Meanwhile, the switching locomotive took its place in serving the steam shovel, but before doing so it assembled a long train of empty cars behind the steam shovel, which were hauled under the derrick, as needed, by a small air hoist mounted on the back end of the steam shovel.

Ring Drilling and Shooting

One of the advantages of using an auxiliary tunnel in conjunction with a main heading is that it facilitates drilling and shooting for the enlargement of the railroad tunnel in parallel transverse rings (ring drilling). In the Moffat Tunnel, drills were set up in the main heading and holes were drilled in a radial direction perpendicular (or with a slight lift) to the axis of the

tunnel. The depth of these holes was such that their bottoms were 6 to 12 in. outside the periphery of the completed tunnel. The holes were shot one ring (or rather two half rings) at a time, furnishing muck for the Osgood shovel to load.

In this method all drilling is done without interfering in any way with the shovel operations. Drilling may be carried on hundreds of feet in advance of the other work without interruptions. Access is obtained to the main heading through one of the cross-cuts, serving for piping, wiring and transportation of workmen, equipment, and supplies. The ring-drilling method contemplates the use of a main heading of minimum size (this being expensive work) and the enlargement to full size in only one more operation, giving a maximum amount of excavation to be handled by the economical air-shovel method. These advantages are important and outweigh the many disadvantages, to be mentioned later.

As a modification of this method it has been proposed to drive the heading at one side of the railroad tunnel instead of on the center line. This would facilitate drilling, in that there would be less holes, and blasting, in that the holes would be of more uniform depth.

It was also proposed to introduce another operation—widening out the heading to full width before the ring-drilling operation was carried on. The object of this was to minimize over-breakage on the sides of the railroad tunnel; to facilitate the drilling of the lower holes in the ring-drilling operation; to diminish the amount of rock to be blasted by the ring method (there being a tendency to have operations blocked by the large quantity of muck to be handled); to separate the bench from the roof, making it easy to shoot the bench and roof separately; to enable a more rational distribution of holes to be drilled in the ring drilling; and to save in the use of dynamite.

However, the only change actually carried out* was to drive the main heading 1 ft. lower than previously planned. This made the nominal bottom line of the heading 7 instead of 8 ft. above the sub-grade of the railroad tunnel, which had the effect of reducing the drilling of "down" holes mainly by making them shorter (there was little trouble drilling the upper ones) and of facilitating the breakage of the rock in the bench in the blasting operations. There was enough over-break in driving the main heading to furnish the necessary clearance for handling drill steels. The blasting of the roof was always comparatively easy, everything being in plain view with no muck in the way.

A great deal of experimenting was done by the contractors' organization in bringing this method to its final state of satisfactory operation. At the outset it was apparent that the shooting could not be done in complete rings. If there were no muck pile in the way, "fly" rock would be thrown far down the tunnel, making an endless task for the shovel in picking it up. On the other hand if a muck pile were allowed to accumulate high enough to stop the fly rock, which was absolutely necessary, then muck would fall down into the heading and cover several rows of holes not yet blasted.

* Other developments later took place; it is expected that this will be brought out in subsequent discussion.

Thus it was necessary to shoot the bottom part of the ring in advance of the top a distance of 16 ft., or four rings. This, in turn, introduced a bad condition at the sides of the main heading where the holes were short and did not have a good face against which to break, so that there was a tendency for rock from these holes to fly across the tunnel to the opposite side. There was also a tendency for muck to fall down in front of the lower bench in excessive quantities, leaving no place to which the rock from the lower bench could be thrown as the result of the blasting. This was especially true where the rock was relatively soft; frequently it became necessary to shovel away by hand a quantity of muck lying on the bench in order to load and blast the bottom holes.

It was customary to blast two rings at a time, that is, two upper half rings and two lower half rings. All the holes in the upper half rings were blasted in succession by the use of delay exploders. In the lower rings or bench the first row was shot with instantaneous or "no-delay" exploders and the second row with delay exploders. In order to throw the muck away from the bench as much as possible it was important to use instantaneous exploders for the first row, so that the explosion of all the holes would be simultaneous. Under these circumstances it was the usual program to have such an accumulation of muck in front of the bench at the end of a week or ten days that it was necessary to stop blasting additional rock and allow the shovel to clean up as closely as possible to the bench and then start all over again. Instances of missed holes and consequent "high bottom" of course would necessitate an immediate cleaning up. The bottom holes were loaded heavily, while the top holes were loaded with little dynamite.

The successful drilling of the rings was quite a problem particularly the drilling of "down" holes. It is absolutely necessary in drilling to have all the holes in the adjacent rings parallel and to have the holes in any one ring uniformly spaced and drilled to a depth at least 6 in. outside the periphery of the tunnel. The bottom holes are drilled at least 1 ft. outside.

In drilling "down" holes, heavy two-man drills of the jack-hammer type were first tried. These drills were so heavy to handle that it was impracticable to drill a hole in any other direction than approximately straight down, and although a mounting for them was considered, it was never tried. Standard drifter drills mounted on an arm on a column were then tried, but were not at all successful at first because the steel became jammed in the holes. This was remedied by arranging the water needle so that a large amount of air was forced through the drill steel in addition to the water, thus by the use of a home-made air siphon removing the water and rock cuttings from around the hole.

To remedy the difficulty in pointing the holes it was decided that the drills should be mounted on a column arm carried on a transverse horizontal bar (Fig. 11). Mounted on the column arm with the drill was an apparatus consisting essentially of two round disks about 4 in. apart, concentric about the column arm and at right angles to the axis of the tunnel. In these disks were pairs of holes drilled in such a way that a steel pin could be inserted in the two holes of a pair until the pin came in contact with the

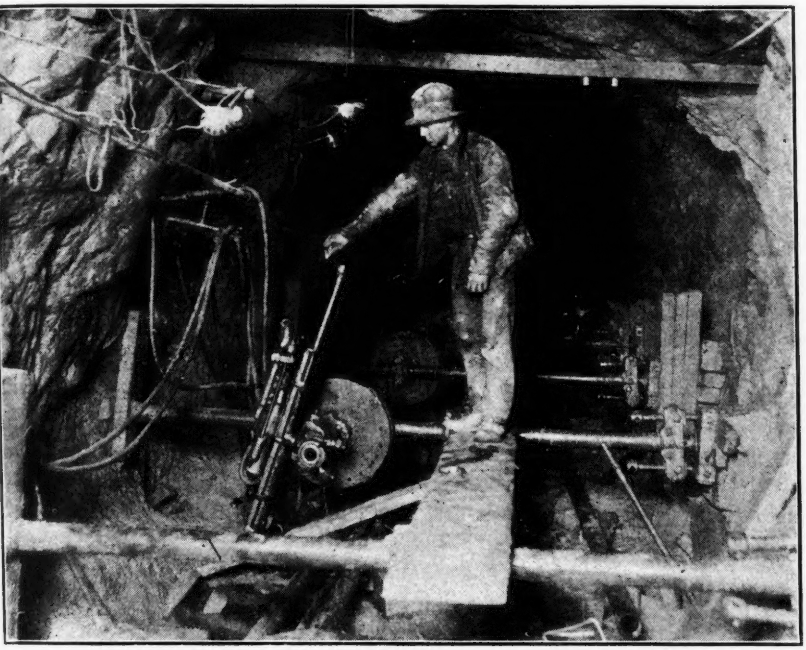

FIG. 11.—RING DRILLING, MOFFAT TUNNEL, EAST PORTAL.

FIG. 12.—RING DRILLING, MOFFAT TUNNEL, EAST PORTAL.

shell of the drill. The proper spacing of these holes gave the drill runner a ready means of pointing the drill. Spacing of rings about 4 ft. apart seemed to give the best results. For this spacing, the horizontal bars were set up 8 ft. apart and, by drilling on a column arm of such a length that the drill could be set up 2 ft. from the bar on either side, all the bottom holes could be drilled.

As shown on Fig. 13, two settings for the column arm were used to drill a complete set of bottom holes. This spacing was necessary in order to obtain clearance for handling the drill steels, and convenient in that it minimized the work of shifting the drills from one hole to the next and also the amount of mucking necessary to get a clear surface of rock in which to start the drill. This latter requirement was quite a problem, for, although the heading had been previously mucked carefully by hand, there was always further accumulation of rock débris, rock cuttings, and rubbish, and necessarily all depressions were always filled with water. In fact, this is one of the great disadvantages of the ring-drilling system, as the cost of this hand-mucking per foot of tunnel is more than one-half the cost of the air-shovel work. After the holes were drilled, they were plugged with a big wooden plug until ready for use.

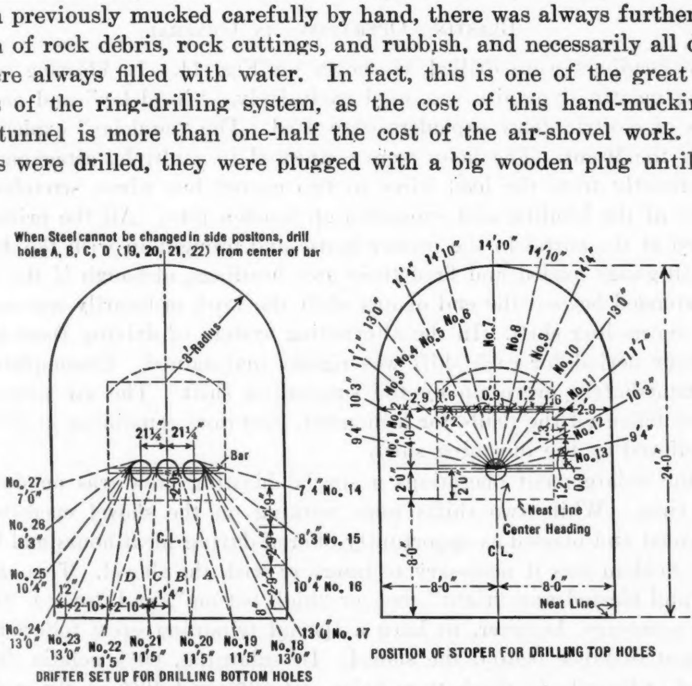

FIG. 13.—RING DRILLING DIAGRAMS, RAILROAD TUNNEL, MOFFAT TUNNEL.

The upper holes of the rings were always drilled with stoper drills. Air-feed stopers proved very satisfactory. As shown on Figs. 12 and 13, all these holes were drilled from one setting of the stoper on the center line. The stoper was mounted on a bar as in the case of the drifter drills. At first the drills were mounted directly on a bar, which was of lighter weight than that used for the drifter drills. This required a bar to be set for each ring. Later, however, the same bar was used as for the drifter drills, and the stoper was mounted on a column arm on the center line of the tunnel. In this way a bar set every 8 ft. sufficed. The pointing of the holes drilled by the stopers was facilitated by placing another bar several feet above the drill bar and

parallel to it, on which were marks to show the alignment for each hole. Even with all these aids for the drill runners in the matter of pointing holes it was soon found necessary to have an inspector on each drill shift to check up and supervise the setting of the drills and drilling of the holes.

After all the experimenting the final methods for the enlargement work were very satisfactory. After lowering the heading 1 ft. and probably also due to the harder and better breaking rock, the blasting operations became quite successful in that it was no longer necessary to stop blasting operations at intervals of a week or more to clean up. Unless there were missed holes the blasting was continuous. The muck-pile was maintained just high enough so that there was no "fly" rock. The drilling and blasting was done so accurately that there was a minimum of over-break and trimming.

Blasting Operations in General

The headings were drilled as shown on Fig. 14. In blasting, 60% low freezing gelatin dynamite was used exclusively. "No-delay" and about four periods of electric delay exploders were used. The "no-delay" exploders were used in the V-cut. The holes were connected in multiple, attachment being made directly from the lead wires to two copper bus wires, stretched across the face of the heading and supported on wooden pins. All the primers were prepared at the portal in the primer house and brought in with the dynamite. The drill gangs loaded and fired their own headings, although if the required time extended beyond the end of any shift the work ordinarily was completed by the succeeding shift. In the alternating system of driving these headings the 8-hour period for each shift was rigidly maintained. Uncompleted tasks were immediately taken up by the succeeding shift. The air pressure was not even taken off the drills for a moment, operators remaining at their posts until relieved by the incoming shift.

In the enlargement operations a special blasting crew was on duty most of the time. While two shifts were working on the shovel operation, this crew loaded and blasted as opportunity offered during meal hours and between shifts. Seldom was it necessary to interfere with the shovel. This crew also drilled and blasted any "tight" rock or "high bottom", and "scaled" the roof. It was necessary, however, to have a special trimming crew follow up at a convenient distance behind the shovel. In enlarging, 60% gelatin dynamite was used ordinarily in the bottom holes and 60% and 40% in the roof holes. These holes were connected in series. Two rings were usually shot at the same time by using no-delay exploders on the first ring and delay exploders on the second. It would be desirable to devise a system of shooting wherein the second ring or a third and fourth also could be shot simultaneously, so that the advantage of the greater effect of such shooting could be obtained.

Ventilation of East Portal

Perhaps the greatest advantage derived from the use of the auxiliary tunnel was its solution of the ventilation problem. Without it the products of combustion of so much dynamite and the heat and bad air caused by the presence of so many workmen could hardly be disposed of satisfactorily. At this alti-

tude the oxygen in 1 cu. ft. of air is only about two-thirds of that at sea level, and men seem to be much more easily overcome by dynamite fumes than at lower elevations.

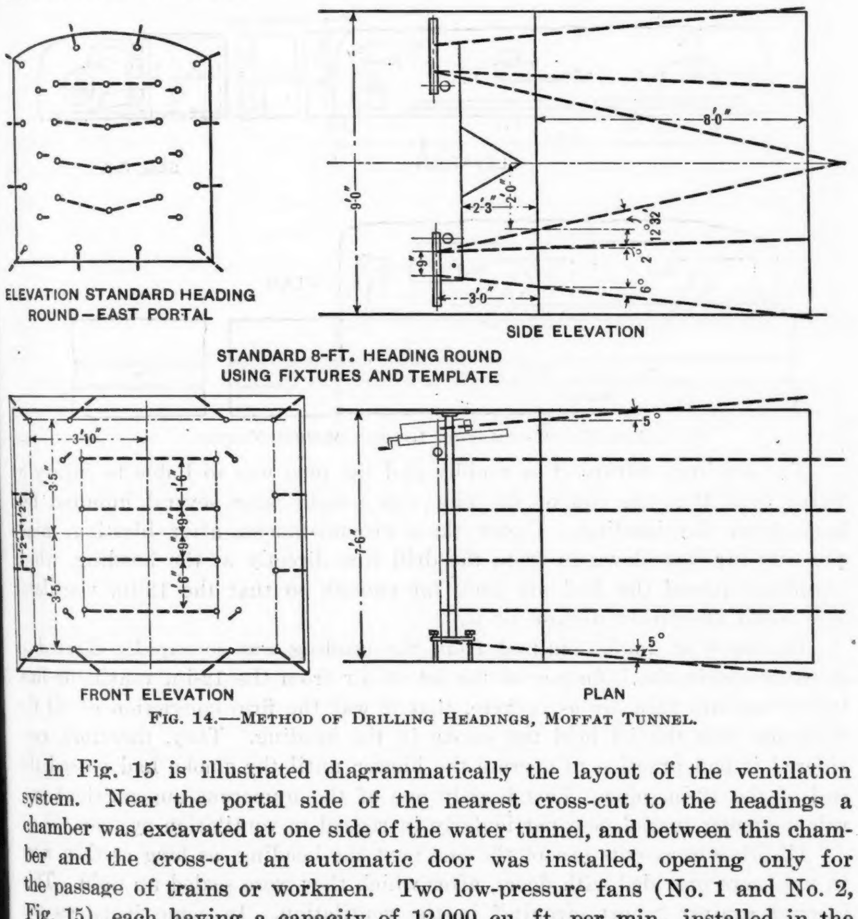

FIG. 14.—METHOD OF DRILLING HEADINGS, MOFFAT TUNNEL.

In Fig. 15 is illustrated diagrammatically the layout of the ventilation system. Near the portal side of the nearest cross-cut to the headings a chamber was excavated at one side of the water tunnel, and between this chamber and the cross-cut an automatic door was installed, opening only for the passage of trains or workmen. Two low-pressure fans (No. 1 and No. 2, Fig. 15), each having a capacity of 12 000 cu. ft. per min., installed in the chamber, forced air from the water tunnel through a duct which opened on the other side of the bulkhead surrounding the door, and thus caused a strong draft inward through the water tunnel, thence through the cross-cut and outward through the railroad tunnel. The pressure against which these fans operated was never measured but obviously it was small.

Also in the same chamber was installed a blower (No. 3) with a capacity of 4 000 cu. ft. per min., designed for a pressure of 4 lb. This blower, working on the plenum system, took air from the water tunnel at this point and delivered it through a 12-in. pipe to a point near the headings of the water and railroad tunnels. By means of valves at the cross-cut the full capacity of the blower could be directed to either heading at will. This blower was operated usually only after blasting in the headings, to force the foul air

back only as far as the first cross-cut from the heading, at which point it could be caught up by the strong current of air from the fans and forced out through the railroad tunnel.

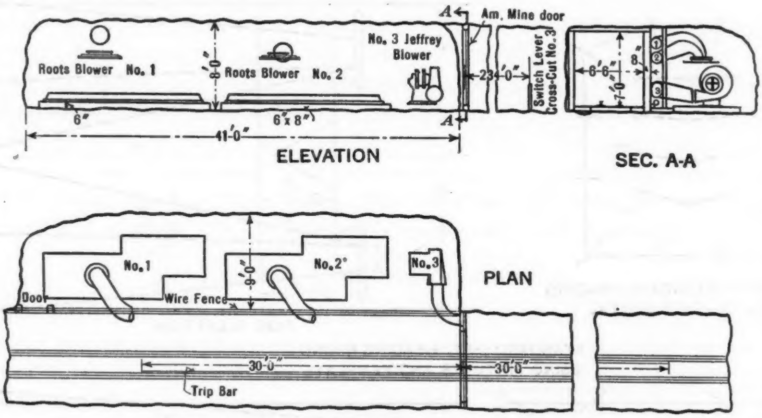

FIG. 15.—VENTILATION LAYOUT, MOFFAT TUNNEL.

The headings advanced so rapidly and the pipe was so liable to injury by flying rock that the end of the pipe was usually kept several hundred feet back from the heading. Under these circumstances, after blasting, high-pressure air was blown in from the drill line directly at the heading, which expedient forced the foul air back far enough so that the 12-in. ventilator line could effectively dispose of it.

The bank of smoke emitted from the heading was so rapidly diluted as it came within the influence of the jet of air from the 12-in. ventilator line, vanishing into thin air as it were, that it was the firm conviction of all the workmen that the jet held the smoke in the heading. They, therefore, considered it bad practice to operate the blower until the smoke had passed the end of the 12-in. pipe. This is only one of the numerous superstitions prevalent among tunnel men particularly in regard to ventilation systems.

All other cross-cuts except the one next the heading, as long as they were in use, were provided with doors, after which they were sealed up tight. This insured against "short circuits" in the ventilation. In some instances the headings were allowed to advance a distance of two cross-cuts before it was necessary to move the ventilation system. In this the controlling factor was the capacity of the blower to force the requisite amount of air through the pipe. This blower was supposed to be able to supply 4 000 cu. ft. of air through about 1 mile of pipe, but leakage made it advisable to move oftener. The pipe was 12 in. in diameter, 20-gauge, with slip-joints. Extreme care was taken to make the joints tight, even by going so far as to put reinforcing rings on the inside in order that a joint of the Dresser type could be used. These connections were effective, but on the other hand it was found that even at the light pressure (4 lb.) all the riveted joints leaked also. In addition, the pipe was noisy, apparently because of the pulsating discharge of the blowers.

Even with this large amount of air for ventilation the condition was not entirely satisfactory in that the completed railroad tunnel between the bench and the portal remained so smoky as to interfere seriously with trimming operations and transportation.

Workmen's Bonus

The principal requirement in all this work was speed. To this end a bonus program for the workmen was early considered. In tunnel work it is very hard to devise a plan that will obtain results and at the same time be fair. On the adoption of the alternating system of working the two headings it was apparent that operations would become so systematized and methods so standardized that a bonus plan could safely be tried. After considerable discussion the following plan was adopted applicable only to the heading crews at the East Portal.* Quoting from the statement announcing the bonus plan, which was started on May 1, 1924:

"The number of crews employed and their organization are to remain as they are at present. No changes are to be made in the distances required due to changes in the character of the rock or for other reasons. The base rate above which the bonus is to be paid is for an average of 17 ft. per day in both the water tunnel and main headings or 34 ft. total. Should the above crews, as they will at times, do work in the cross-cuts, credit will be allowed for work done there but for no other reason such as enlargements for plant.

"The bonus will be paid at semi-monthly periods corresponding to the pay roll periods on the total footage accomplished in these periods, which means that in a 15-day period at 17 ft. per day per heading, the total footage above which a bonus will be paid is 510 ft. and in a 16-day period 544 ft. Furthermore the bonus will be paid only to men who work every shift during the period except when excused by the superintendent on account of injury in which case a bonus will be paid proportional to the number of shifts worked.†

"In case a man is promoted from a lower to a higher group or *vice versa*, as shown below, his bonus will be based on the number of shifts worked in each group.

"The bonus will be a fixed amount to be paid to each man in proportion to the number of feet excavated in excess of the minimum as stated above. The fixed amount, however, is not the same for all men in the crews but varies according to the importance of their duties. For this purpose the men are divided into three groups.

"Group No. 1 will consist of: Assistant Superintendents, Shifters (Foremen), Scalers, Miners (Drill Runners), Helpers (Drill Helpers or Chuck Tenders), and Nippers.

"Group No. 2 will consist of: Mucking Machine Operators and the Muckers in the crew of the Mucking Machine.

"Group No. 3 will consist of: Tunnel Motormen and Brakemen, engaged in hauling from headings only.

"There will be no others than those stated above to be entitled to a bonus.

"The bonus to be paid to members of the different groups for each additional foot of advance will also vary in that it increases with the distance gained as stated below, it being understood that for a 16-day period all distances given below are increased by 34 ft.

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* Later this was used at the West Portal when ground permitted the alternating method, as will appear in discussion of the paper.

† Soon afterward the superintendent also was authorized to issue excuses on account of illness.

"For members of Group No. 1 there will be paid:

"For every foot advance above 510 ft. up to and including 600 ft. the amount of 15 cents per ft., which we will call rate 'A'.

"For every foot advance above 600 ft. up to and including 675 ft. the amount of 20 cents per ft., which we will call rate 'B'.

"For every foot advance above 675 ft., the amount of 30 cents per ft. which we will call rate 'C'.

"For the members of Group No. 2, the distances are the same but the bonus paid is 10 cents, 12 cents, and 18 cents, respectively.

"For Group No. 3, the bonus paid is 5 cents, 7 cents, and 10 cents, respectively."

Judged by the high rates of progress later attained, these rates appear too high.

No attempt was made to apply a bonus to the other operations at the East Portal. The enlargement operation could easily progress faster than the headings by working two shifts per day, and as for the other work, extra progress required only the addition of more men or equipment.

Disposal of Spoil

The matter of disposal of the material excavated from the tunnel was not very serious. The muck cars from the water tunnel were dumped into chutes leading to 4-yd. cars standing in the approach cut to the railroad tunnel. A whole train was dumped without shifting the cars. These cars were hauled out on the spoil bank by an 8-ton trolley locomotive which also served the railroad tunnel. The spoil bank was located as to line and grade to form a convenient permanent railroad approach to the tunnel.

Mechanical Equipment

The compressor equipment at the East Portal consisted of one compressor with a capacity of 450 cu. ft. of free air per min., two of 1 150 ft., one of 1 245 ft., and one of 1 300 ft. At times, all these machines were in use. It is to be remembered that at this altitude a large allowance in extra capacity had to be provided. The main high-pressure air line was $7\frac{5}{8}$-in. well-casing, with Dayton joints for about $1\frac{1}{2}$ miles and $6\frac{1}{4}$ in. for the remainder. Three of the large compressors were driven by 200-h.p. synchronous motors which served to maintain the power factor. The monthly maximum demand average of three 3-min. peaks up to August 1, 1925, was about 1 100 h.p. This, however, did not include one of the large compressors, which was put in operation later.

All drill steels were used only once, requiring the sharpening of about 1 500 bits per day. Two sharpening machines were used. The steels were heated in oil furnaces, and another furnace was used for tempering. All the furnaces were served by special low-pressure blowers to avoid the waste of high-pressure air.

Four-point bits were used, with extra wing thickness to avoid rifling of the holes. Later, the so-called McClellan bit, a modification of the standard four-point bit, was adopted with satisfactory results.

Storage battery locomotives were provided with extra batteries so that they might always be ready for service. A motor generator set supplying 250-volt direct current and located in the power house was used for charging

the batteries and also for trolley service, supplemented as heretofore described by motor generator sets under ground.

Three 400-kv-a, single-phase, 44 000-2 300-volt transformers occupied the East Portal Sub-Station.

Plan of Operations at the West Portal

On account of the great amount of "heavy ground" at the West Portal the operations there were much different from those at the East Portal On beginning the work little was known of the amount of this unsound rock and it was assumed to extend less than 1 800 ft., so that the plant and equipment provided was practically the same as at the East Portal.

The portal cuts were entirely in earth at the West Portal, likewise the water and railroad tunnels for a short distance. The water tunnel was given the same relative location with respect to the railroad tunnel as at the East Portal. It was started first and was a long way underground before the portal of the railroad tunnel was turned. Work on the portal cut of the railroad tunnel was slow on account of difficulties with the shovel, with frozen ground, and with disposal of the excavated material. Realizing this situation, the first cross-cut from the water tunnel to the railroad tunnel was excavated as near the portal as possible. Although every one continued optimistic as to the kind of material about to be encountered, no really sound rock was found for many thousand feet.* Under these circumstances the use of the water tunnel as an auxiliary heading was a tremendous advantage in expediting the work by providing access to the railroad tunnel through several cross-cuts at the same time. In this way it was possible to carry on excavation work in the railroad tunnel in as many as eight different places at one time.

Water Tunnel in Heavy Ground, West Portal

While the idea of "bad ground" immediately conjures up ideas of high cost and small progress in tunnel work, this is not strictly true in the case of a tunnel as small as the water tunnel and in rock of the particular character encountered at the West Portal. These troubles are primarily due to the necessity of timbering close to the working face and of taking precautions to avoid knocking down the timber in blasting operations. In the water tunnel at the West Portal, however, where timbering had to be carried close to the working face, the rock was uniformly so soft as to need little drilling, thus saving labor, and little dynamite in blasting, thus saving this large expense. The cost of timbering, however, was a large item. Much of this tunnel was excavated without the necessity of timbering close to the heading. Heavy ground pressures ordinarily did not become evident until some time after the face was exposed.

Where necessary, the water tunnel was timbered with square sets. Usually the caps and posts were of native lodgepole pine about 10 to 12 in. in diameter. In much of this tunnel it was necessary to drive ahead for a new set by driving spiling or poling-boards of 4-in. native timber. After the poling-boards had been driven, a cap was set under the front ends to support them,

* See later discussion for the extent of this unsound rock.

the cap in turn being supported by two cantilever beams extending from the under sides of the last caps in position. Under the protection thus afforded mucking operations were carried on so that posts could be set in position under the ends of the advanced caps.

A swinging false set for this operation was suggested but never used. No doubt it would have been an improvement for the worst of this work as it would have facilitated greatly the driving of the poling-boards. In part of this tunnel it was also necessary to drive poling-boards on the sides. In some favorable places the timber could be erected at a distance back from the heading so as not to interfere with the excavation. Then, again, much of the water tunnel did not require timbering at all. The uncertain nature of this rock is illustrated by the fact that from Cross-Cut No. 3 for 3 433 ft. eastward there were twenty places needing timber, aggregating only 1 167 ft. in all. The peculiar feature of the heavy ground at this end of the water tunnel was the relatively high pressure developed on the sides of the tunnel. Because of this pressure there was always a tendency for the posts to be forced into the tunnel. Many of them were broken and it became necessary to place reinforcing sets, frequently of 12 by 12-in. Oregon pine.

At first the posts were set vertically, a spreader of 3 by 8-in. native timber being used at the bottom between opposite posts. Settlement of the posts frequently broke these spreaders. Afterward it was decided to set the posts battering outward so that their bottoms were 12 in. farther apart than their tops. After this, no more trouble was experienced from posts "kicking in"; but by this time the worst of the "heavy ground" was past. Many caps also were broken and were replaced with Oregon fir sets. In addition, many of the posts settled, bringing the caps down with them and thus decreasing the clearance overhead to a point requiring the replacement of the timber at a higher elevation. These serious troubles only extended about to Cross-Cut No. 3 and beyond that point, generally speaking, the timber stood very well. Cross-Cut No. 2 itself was so bad that it was necessary finally to reinforce the timber by using steel I-beams for caps. Even then the clearances were cut down to such an extent by these I-beams and other reinforcing timbers as to interfere seriously with its use.

Although much of the first 8 000 ft. of the water tunnel did not require any timbering, the adjacent railroad tunnel required it all the way.* Beyond Cross-Cut No. 3 the average character of the rock in the water tunnel improved greatly, containing considerable stretches of fairly good material, but, at intervals, decidedly crushed zones that were the occasion for very serious cave-ins.

The use of native timber in the water tunnel was unsatisfactory in another important respect in that it was peculiarly subject to fungus attack and rapid decay. With the improved ventilation resulting from the use of the water tunnel as an intake, as at the East Portal, this condition was greatly relieved.

At first the water tunnel at the West Portal was driven independently of any other headings. The shifts of men assigned to this heading were selected

* For very heavy ground beyond this point see later discussion.

by promotion from other work, the object being to expedite this part so that the ground could be quickly explored and more cross-cuts opened up for access to the railroad tunnel and thus the work "holed through" as soon as possible. In most of this work hand-mucking methods were used and the drill shift not only did its regular work of drilling and blasting but also all the timber work with the help of the muckers. Later, a Conway loader was used in this heading but it was of doubtful advantage in passing through bad ground as there was much lost time anyway.

The drilling and blasting followed the method as used in independent headings already described, namely, with two drills only, mounted on a horizontal bar. After July 1, 1925, however, the rock then being of much better quality, the heading was driven by the alternating method as at the East Portal, in turn with the main heading, then being driven from Cross-Cut No. 6. Thereafter progress in these headings became almost as good as at the East Portal.

Timbering Railroad Tunnel in Heavy Ground, West Portal

By far the most interesting of the operations on the Moffat Tunnel was the excavation and timbering of the railroad tunnel. The decision to try to hold this heavy ground permanently with timber only was a bold one. There is no record in engineering literature of any one having ever seriously considered such a thing as practicable, nor is it likely that it will ever be attempted again.

The ideas in regard to the weights and pressures that a timber or masonry tunnel lining will have to sustain as evolved by various theorists, seem to be in no way in accord with any reasonable assumptions and quite impractical in their applications. There is also the question of so-called "swelling ground" *versus* just ordinary "heavy ground". What, if anything, is there in the theory current in mining districts that the weight on the timbering can be relieved by "bleeding" the timber, that is, raking out some of the loose rock behind it? Or what in the theory that as time goes on weight on timber grows less instead of greater, due to the fact that the loose ground outside the tunnel tends to adjust itself into the form of an arch? Are vertical posts less likely to be pushed in at the bottom if set straight, or battered out, or battered in; and will setting the posts on foot-blocks or wedges inclined outward prevent them from "kicking" in? Is it better practice to pack the spaces outside the timber with cordwood or with rock fragments? Which is the better method in a single-track railroad tunnel, to use 3-piece, 4-piece, or 5-piece segmental arches; and what should be the particular design of these arches? How about wall-plates? Is it feasible to build a tunnel in heavy ground without them? Should lagging be made thick enough to resist decay as long as the heavy timbers or should it be made so thin that it will be the first thing to break under load? Is there anything in the old theory that the first requirement in tunneling in bad ground is to hold everything absolutely in place?

The contract for the construction of the Moffat Tunnel showed as a typical section of timbered tunnel a 3-piece set of 12 by 12-in. timber, a 12 by 12-in.

wall-plate, and 12 by 12-in. posts. Timbering was not considered at that time as a serious difficulty and there were elastic provisions in the specifications for a change in design if that became necessary. All timbering operations described in the specifications contemplated the use of side-drifts in which the wall-plates were to be set in place ahead of the face of the heading. This method involves the use of a top heading and the removal of the remaining bench in one or more lifts. To work in well with this plan, the location of the water tunnel at an elevation 8 ft. below the top heading, is rather awkward. As it turned out the best method was to drive the cross-cut on a level grade and then ascend in the railroad tunnel as rapidly as possible.

However, for other reasons it turned out that practically none of the wall-plate drifts was driven. This method was tried in the earth part of the upper heading at the portal but was soon abandoned on account of its slowness. At about the same time, Cross-Cut No. 1 having been finished, a so-called timbered main heading on a rapidly ascending grade was started each way from this cross-cut, and after two or three wall-plates had been set at the portal and the heading permanently timbered to the end of them, a similar main heading was started to meet that coming from Cross-Cut No. 1. This was only a short distance, as Cross-Cut No. 1 was close to the portal.

This timbered main heading was similar to the timbered heading driven in the water tunnel except that the posts were much longer. The dimensions of the heading as to height were governed by the fact that the under side of the cap of the square-set as it was to be left in place must clear the cap of the permanent timber, a nominal clearance of 1 ft. being allowed. The bottom of the heading was low enough to facilitate the placing of wall-plates. The operations of widening the heading and setting permanent segmental timbering followed the driving of the main heading at a convenient distance behind, sometimes not for months afterward.

The first experiment in the way of setting wall-plates from this main heading was to drive them, as it were, laterally rather than longitudinally. To this end cantilevered poling-boards were driven out laterally from the main heading to clear the tops of the wall-plates. This was not successful and was soon abandoned in favor of a method, the so-called "winging out" method (Fig. 16), wherein, working longitudinally of the tunnel, complete 5-piece sets of temporary timber (false sets) were placed outside the prospective position of the permanent timber, ample clearance being left for the placing of the permanent timber. When 3-piece permanent sets were used, these false sets were practically 3-piece sets themselves, the lower segment being quite short and vertical. When the permanent timber was changed to 5-piece sets, the lower leg of the false sets was lengthened but was still left vertical. These false sets were largely used throughout the timbered portion of the railroad tunnel. Sometimes the ground was so stable that the wing sets could be left out if the permanent timber were placed immediately after the excavation of the rock; and sometimes the rock was so bad that only by the most careful work and the extensive use of poling-boards on the top and sides could the widening be accomplished.

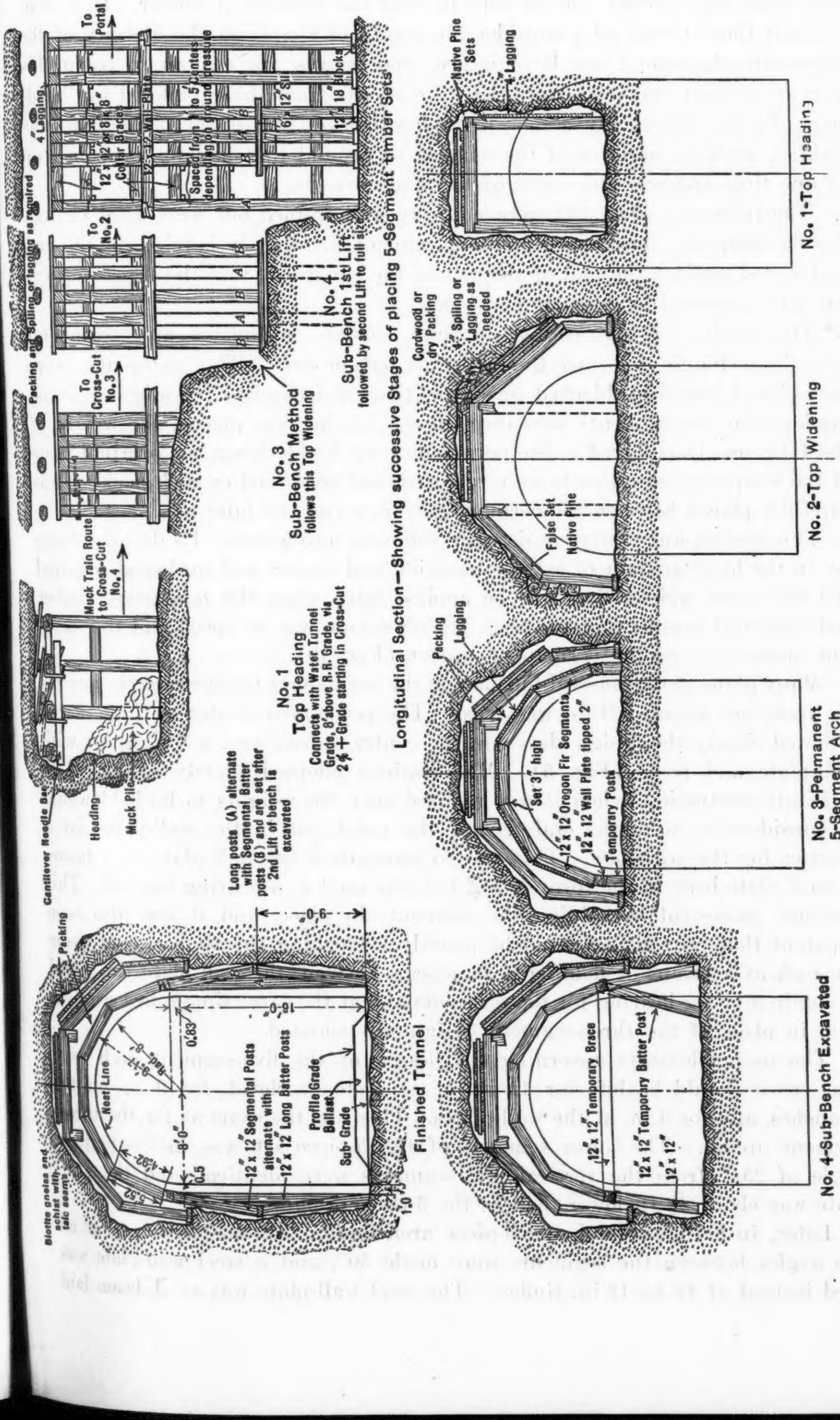

FIG. 16.—TIMBERING METHOD, WEST PORTAL, MOFFAT TUNNEL.

At the cross-cuts the main headings were driven at a steep up-grade until they were high enough for the caps to clear the permanent timber. In widening out there remained a considerable length of tunnel in the vicinity of the cross-cuts that could not be enlarged, and it was the custom to leave this part to the last and work toward the cross-cuts from the completed top headings, driving practically a new main heading on top of the old. This was ticklish work on account of the amount of ground that had to be opened up at one time and was the cause of two bad cave-ins.

The false sets (Fig. 16) were not very well framed but were good enough for the purpose. The upper inclined segment was merely beveled on the end and rested against the end of the cap of the square-set and the short vertical leg was supported by a small foot block.

The setting of the permanent arch timber—wall-plates and segmental sets—immediately followed the widening-out process. The wall-plates were first placed, carefully blocked up in position, and checked for alignment and grade; then the segments were erected, the lagging was placed, the weight of the false sets transferred to the permanent sets by blocking, the original posts of the temporary square-sets were removed, and cordwood or rock packing was carefully placed between the permanent timber and the false sets.

This system was pretty consistently followed throughout. Its disadvantages lay in the large amount of extra excavation and timber and cordwood required and the extra weight that will be applied later when the temporary timber and cordwood compress and decay. Its advantage was its speed and the excellent appearance of the permanent timber (Fig. 19).

Many plans were proposed for placing the permanent timber directly against the rock, but none of them was tried. The permanent timber as first placed followed closely the design shown in the contract drawings, a 3-piece set with wall-plate and posts (Fig. 5). Modifications adopted shortly afterward to facilitate excavation, when it was realized that the weights to be held would be considerable, were the omission of the notch cut in the wall-plate for a bearing for the segments. This was to strengthen the wall-plate as a beam. A steel plate bent in the form of a Z-bar was used as a bearing instead. This was not successful in holding the segments in place, and it was also soon apparent that no reliance could be placed on blocking the timber away from the rock as a means of holding it in place. Instead, the wall-plate was tilted to obtain a better bearing for the segments and at the same time a five-segment arch in place of the three-segment design was adopted.

The main elements governing the design of the five-segment arch were that space should be left for 12 in. of concrete overhead, for 9 in. on the haunches, and for 6 in. at the wall-plates. This was the same as for the three-segment arches. The lower segment of the 5-piece set was inclined at an angle of $25\frac{1}{2}°$ from the vertical, the segments were identical, and the wall-plate was about $2\frac{1}{2}$ ft. lower than in the 3-piece design.

Later, in better ground, the 5-piece arch design was modified in that all the angles between the segments were made 30°, and a steel wall-plate was used instead of 12 by 12-in. timber. The steel wall-plate was an I-beam laid

on its side. Its advantage is that it does not compress, holds the posts and segments in position, and practically puts the segments and posts in contact.

Details of Operations, West Portal

The excavation of the main headings advanced much more slowly than that of the water tunnel. This was due largely to their greater height. At times several of these headings were being excavated simultaneously, access being obtained from the various cross-cuts. Usually only the most advanced heading was equipped with a Conway loader. Access for the heading-widening operations was also obtained from the various cross-cuts. The headings were always driven through from one cross-cut to the other before widening was started in order that circulation of air could be established as soon as possible.

The transportation system in the water tunnel operated in the same way as at the East Portal. The trolley, however, served only the water-tunnel heading and the work in the railroad tunnel, which was carried on from the next cross-cut to the heading. Necessary haulage from other cross-cuts was handled all the way to the spoil bank by storage battery locomotives. The trolley was always given the right of way.

All water from the headings drained out through the water tunnel all the way to Cross-Cut No. 1, where it was discharged into the railroad tunnel. The quantity of water handled probably never exceeded 200 gal. per min.

On the assumption that there was to be but little unsound rock at the West Portal, the same equipment for the enlargement operation to full size was ordered as at the East Portal, an Osgood air shovel, dump cars of nominal 4-yd. capacity, 3-ft. gauge track, 40-lb. rail, and 8-ton trolley locomotives.

In order to have enough head-room for the shovel it was necessary to take out the full bench, nominally 16 ft. high, in one operation and also to keep a clear space 16 ft. wide at all times. In a timbered tunnel this introduced quite a problem as to the temporary support of the overhead timber while excavating. All sorts of travelers were proposed for this work, all supported on tracks at invert grade. The excavation, however, was started at the portal by removing only a small part of the bench at a time, batter posts close to the bench being set to support the wall-plates until permanent posts could be placed.

The rock at this point was comparatively good, a partly disintegrated pegmatite extending in for about 500 ft. from the portal. In attempting to use the shovel it was found to be a very poor piece of apparatus for mucking out short rounds. On account of the small radius of the dipper arm anything in the nature of a corner could not be reached at all. This necessitated much mucking by hand—and into high cars at that. The bench being so high and there being so little head-room above, most of the holes drilled were "lifters".

Needle-Bar Method for Bench Excavation

In any kind of industrial work the first concern is so to organize the work that all men are fully employed for the entire shift, that they do not interfere with one another, and that, if there is a regular cycle of operations each step of

which has to be done in regular rotation, the cycle may be completed at regular intervals of 24 hours each so that the various shifts can go to work at the same time each day. The use of a "call" shift should be avoided on account of the ensuing extra expense, irregular sleep and meals for the workmen, and the consequent demoralization. Particularly should this be avoided where highly trained workmen, such as steam-shovel operatives, are employed.

Under the conditions at the West Portal it will readily be seen that the bench excavation by this method was very unsatisfactory as, among other things, the shovel crew could clean up all the muck from one round in about two hours. This constituted a day's work for them and, furthermore, this work had to be arranged to come during the day shift.

To remedy this situation it was imperative that some better method for supporting the wall-plates be found so that a longer round could be taken out. The solution of this difficulty for the time being was found in the use of two 24-in. by 25-ft. I-beams as so-called needle-beams.

As shown in Figs. 17 and 18, these beams were disposed longitudinally of the tunnel, one on either side. They were supported on transverse cross-beams one of which was in turn supported on the wall-plates at a point to the rear of the permanent posts, and the other on the wall-plates at a point where these plates in turn were supported by the bench not yet excavated. Another set of three or four cross-beams was hung by heavy bolts from the needle-beams. Being placed under the opposite wall-plates, this set served as their only support for 10 or 15 ft., or more. Later, 40-ft. instead of 25-ft. beams were used.

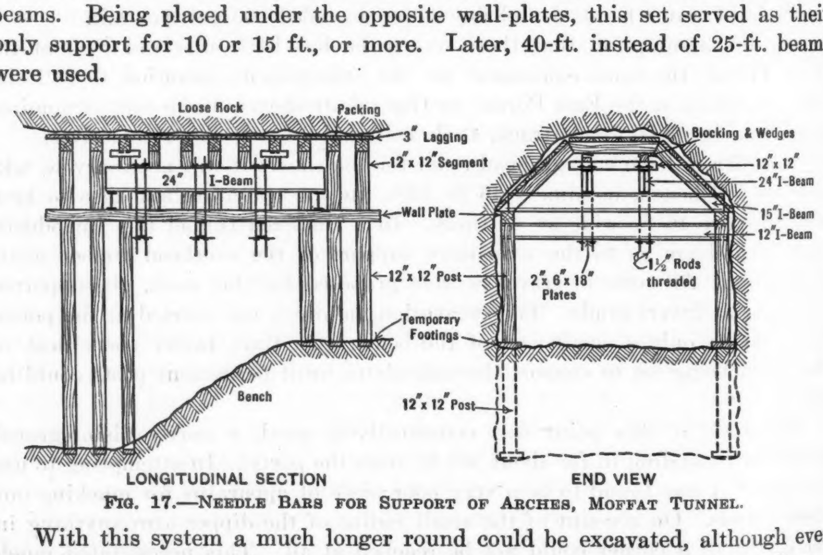

FIG. 17.—NEEDLE BARS FOR SUPPORT OF ARCHES, MOFFAT TUNNEL.

With this system a much longer round could be excavated, although even then it was not possible to employ the shovel crew for its full time. It was also apparent that the needle-beam apparatus should be improved, especially as to the arrangements for moving ahead. Of course it would not be possible to use such a system if heading work was going on at the same time, as the heading would be blocked. In the Moffat Tunnel, however, as all the heading work was done from the water tunnel and cross-cuts, this objection did not hold.

Sub-Bench Method for Extra Heavy Ground

The excavation of the bench went on as described to the end of the pegmatite formation. This point was also the beginning of the worst section of tunnel. In entering this formation the same system was continued but on blasting the first round there was a bad cave-in from behind the wall-plate on one side. The wall-plate settled also and there were indications of careless work in supporting these plates.

It was decided that it was unsafe to attempt to take out such a high bench in such unstable material. The next decision was to take out the bench in two lifts, the so-called sub-bench method. It was also apparent at this time that a concrete lining should be placed in such stretches of unsound rock and that any attempt to depend on timber as a permanent lining would be a mistake.

The method adopted for prosecuting the work as far as it was necessary to use the sub-bench method was first to take out the top lift or upper sub-bench as far as Cross-Cut No 2, the work being handled from the railroad tunnel portal. After passing Cross-Cut No. 2 the upper sub-bench would be handled from that cross-cut and the lower sub-bench started and handled in turn from the portal. On account of the lack of room in camp for more men and the desire to push the heading work there was no attempt to work both sub-benches at the same time, although this could have been done.

The method of excavation was to be by hand, although to cheapen the mucking operation a plan was contemplated of excavating first through the upper sub-bench a trench about 10 ft. wide and 8 ft. deep, the mucking to be done with a Conway loader. There being no loader available, however, this method was not tried.

In timbering the upper sub-bench it was necessary then to design a system of timbering that would enable the lower sub-bench to be excavated readily, and at the same time to provide for a stronger concrete lining than had heretofore been contemplated, one adapted to resist side pressure on the tunnel. At first the idea was to set vertical round posts under the wall-plate at every alternate segmental arch. The temporary posts were set on foot-blocks. At the other segmental arches a permanent post of 12 by 12-in. Oregon pine* was set battering outward. These posts were supported on a 6 by 12-in. continuous sill of Oregon pine. The inside face of this sill cleared the line of temporary posts.

Soon after the excavation had been made, however, heavy side pressure began to develop, and this was provided for by setting the temporary posts battering inward and putting heavy 12 by 12-in. spreaders between the wall-plates at their ends, or about 16 ft. apart. These spreaders soon indicated that the side pressure was increasing, as some of them were indented into the wall-plates ½ in., or more, and the wall-plates were bent in between the spreaders. Later, additional spreaders of 6 by 12-in. Oregon pine were placed intermediate to the first ones. The segmental timbers that theretofore had shown very little indication of being stressed before the bench was excavated,

* This timber has been replaced by separate sets of 12 by 18-in. Oregon fir (p. 204).

also began to take weight, not so much however as the posts under the wall-plates. The sudden increase in vertical weight is easily explained by noting that the weight previously had been taken by the false sets but that now, the foundation having been taken away from under the false sets, all the weight was transferred to the permanent timber. The peculiar design of the false sets is probably responsible for the development of excessive side pressure. With evidence of such tremendous lateral pressure it was quite evident that no ordinary system of segmental timber arches could withstand the strain and that the problem required a heavy concrete lining as its solution.

Meanwhile the completed timber work near the portal began to show evidence of much strain. This was in the pegmatite formation. All the timber here was of 3-piece, segmental arch design. Some of the posts were broken and all of them were bent quite visibly; in two places the wall-plate was broken, rotated over backward, the lower legs of the arches were pushed outward on top of the wall-plates, and the wall-plates forced into the tunnel. To meet this condition reinforcing timber was placed inside the regular sets. This was intended to be removed and a concrete lining placed later.

In November, 1924, the upper bench having passed Cross-Cut No. 2, preparations were made to start work taking out the lower sub-bench. The ground beyond Cross-Cut No. 2 having become more stable as the work advanced, it was decided to drop down to the bottom grade and take out the full bench, using hand-mucking methods. At Cross-Cut No. 4 also, work was started taking out the full bench, working west from the cross-cut and using hand methods. At both Cross-Cut No. 2 and Cross-Cut No. 4 the method was changed somewhat, as soon as a sufficient height of bench was reached, in that the bench was taken out in two lifts, the upper one being 20 ft. or more ahead of the lower. This was because more men thus could be put to work advantageously.

By means of wheel-barrows on a runway the muck was carried out and dumped into cars standing on the lower level exactly as is done in mucking out by hand a tunnel driven by the top-heading and bench method. Wall-plates were held successfully in place by the use of 10-ton single-screw pipe-jacks extending transversely of the tunnel and tightly jacked against opposite wall-plates. It is only necessary to use a sufficient number of such jacks, placed with judgment, to hold any overhead weight.

On the lower sub-bench no progress had been made since the previous July (1924). Slow progress was anticipated here and to expedite the mucking operation a so-called St. Joe loader was obtained on trial. It was operated entirely by electricity and would load into the high 4-yd. muck cars. After a trial extending over a week or more it was decided that this machine offered little advantage over hand-mucking methods mainly on account of the lack of clearance for its quick operation. As great hopes had been held out for the successful operation of this machine no other adequate method of mucking was available and it was necessary to return to hand-mucking methods.

The first timbering in the excavation of the lower sub-bench consisted in removing the temporary round posts placed at the time the upper sub-bench

was removed, and replacing them by full length 12 by 12-in. posts extending to sub-grade and set on foot-blocks. Also, under the 6 by 12-in. sill was placed a short post battering in so that at the bottom it was in line with the long posts. This short post was set under the permanent batter posts placed with the upper bench and in connection with them was intended to form an arch that would help to resist side pressure. However, mainly on account of the compressibility of the 6 by 12-in. sill in resisting compression across the grain they were of little value in this respect.

Immediately on excavating the lower sub-bench, the side pressures increased greatly. It was necessary to place spreaders lower down than the wall-plates and directly against the posts, which showed signs of breaking. A trussed design for the long posts adapted to take side strain was adopted and it was decided to replace the segmental posts by trussed posts also.

Traveling Cantilever Needle Bar

At this time a policy was adopted of attempting to hold all the tunnel by permanent timber, there being little realization of the tremendous pressures to be withstood and the inadequacy of timber for such a purpose. It was also determined to try out the Osgood shovel again. For this purpose it was necessary to return to the needle-beam method. A modification of the original needle-beam method for supporting the wall-plates was adopted (Fig. 19), using the cantilever principle. Two heavy 42-in. girders, 60 ft. long, cantilevered out from the high bench on which they were supported, were the main feature of the design. These girders were framed together in one unit and were pulled ahead on a track by a small air hoist. Improved cross-beams and a wedge arrangement for raising them to a bearing on the under side of the wall-plates were also features of the method. These cantilevers were designed for use on the full bench, which being at this time east of Cross-Cut No. 2 could not be reached by the Osgood shovel because the lower sub-bench was not yet excavated.

It was then determined to use the cantilevers and the Osgood shovel on the lower sub-bench. This necessitated the erection of the girders on cribbing about 8 ft. high above the sub-bench (Fig. 20) and the removal of the spreaders between the wall-plates. It was hoped that it would be safe to take out the spreaders and use instead 3-piece arches placed alternately between the 5-piece arches already in place. These 3-piece sets were intended to maintain the wall-plates against lateral pressure as did the spreaders and it was necessary to make the substitution in advance of the progress of the girders. The clearance under these 3-piece sets was less than under the 5-piece sets, so it was necessary to lower the girders which, in turn, made it necessary to lower the sub-grade of the tunnel 9 in. to provide clearance for the shovel in loading cars.

Original Timber Lining Replaced by Separate Sets

Through the worst of this heavy ground section trussed posts were placed on close centers and for part of the way they were placed practically "skin tight". All this work turned out to be inadequate for shortly after the timber

was finished tremendous pressures began to develop. Again the wall-plates were the main source of trouble. Even with the thorough bracing against side pressure provided by the 3-piece reinforcing sets the wall-plates were gradually forced into the tunnel. In so doing they were crushed and sheared in places almost to shreds (Fig. 21). The trussed posts were also inadequate to withstand the pressures and spreaders were again placed in part of the tunnel. An attempt was made after Cross-Cut No. 2 was reached to turn the drainage of the water tunnel into the main tunnel. Immediately some of the posts on the south side of the tunnel "kicked" in at the bottom, owing to the softening of material around the foot-blocks by the water.

It was now realized that the timber already placed was inadequate and would have to be replaced by something much stronger and particularly that the wall-plates must be eliminated.

The design adopted which is about the last word in timbering consists of a 3-piece arch set of 12 by 12-in. timber, the inclined legs being placed somewhat more vertical than usual. These sets rest directly on 12 by 18-in. posts. To erect this timber it was necessary to remove all the old timber and much of the cordwood packing behind it, just enough ground being exposed at any one time to enable one new set to be erected. This reconstruction work will probably extend over at least 2 000 ft. of tunnel much of the timber being set "skin tight". Later, it was decided to make the inclined legs of the arches of 12 by 18-in. instead of 12 by 12-in. timber, the 18-in. dimension, as with the posts, being transverse of the tunnel. Meanwhile the bench excavation had been making progress and east of Cross-Cut No. 2 the full height of bench was attacked. This stretch of tunnel, although consisting of rock much better than that where the sub-bench method was used, had to be excavated with great care to avoid cave-ins due to the great height of the unsupported bench. As further progress was made, however, the rock became more self-supporting.

Ventilation of West Portal

The ventilation system installed at the West Portal was the same as that at the East Portal. It did not work so satisfactorily, however, due to the main headings being late in "holing through" and to smoke sifting through from the railroad tunnel to the water tunnel through some of the cross-cuts which could not be made air-tight on account of being timbered. The other apparatus used at the West Portal was much like that already described for the East Portal. There was not, however, as much air-compressor capacity required as at the East Portal because less drilling was required.

Disposal

At the West Portal all muck trains from the water tunnel went directly to the dump, descending on a 2% grade to the lower level.

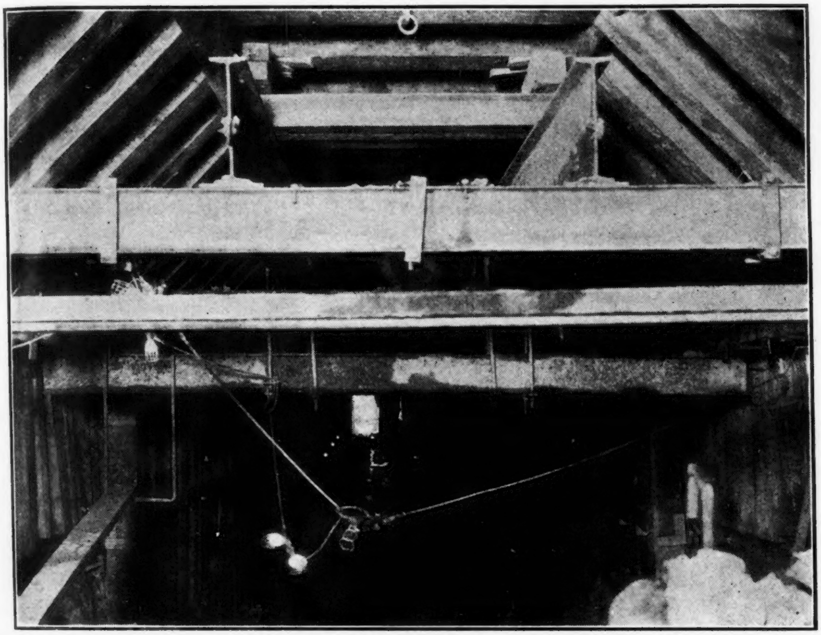

FIG. 18.—MOFFAT TUNNEL, WEST PORTAL, SHOWING I-BEAM NEEDLE-BAR AS SEEN FROM HEADING.

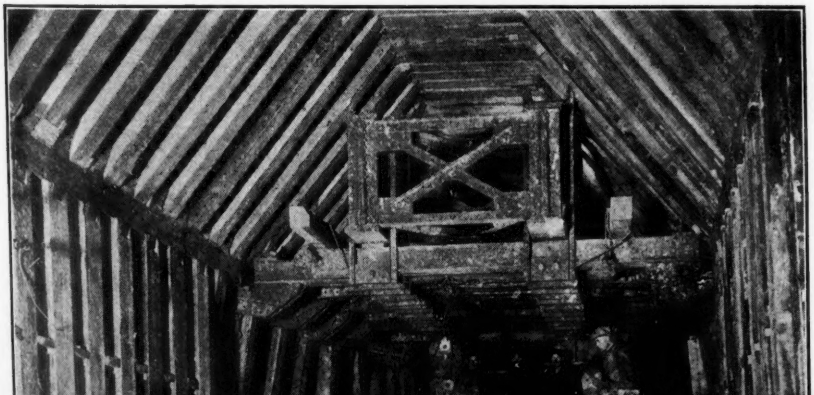

FIG. 19.—MOFFAT TUNNEL, WEST PORTAL, SHOWING LEWIS TRAVELING CANTILEVER NEEDLE-BAR, LOOKING TOWARD HEADING.

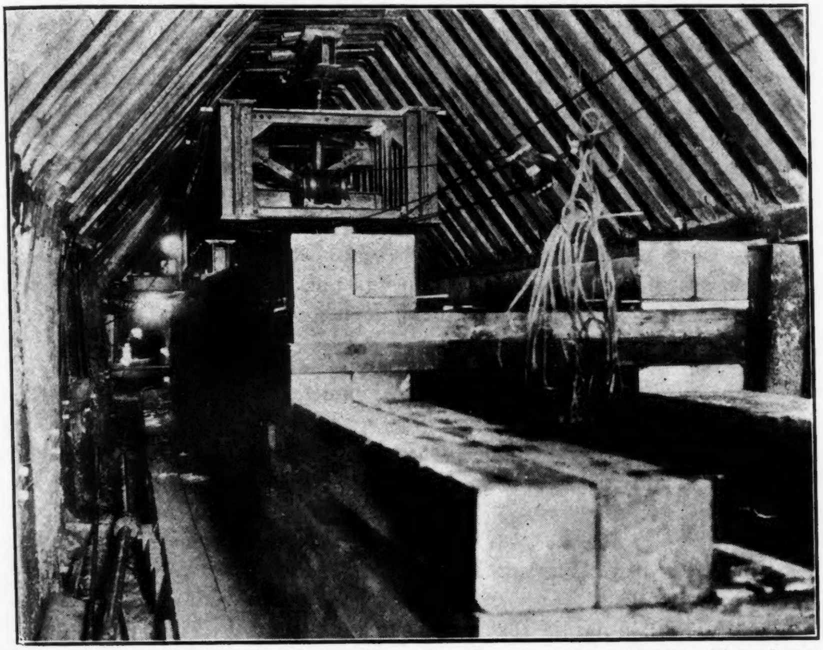

FIG. 20.—VIEW OF CANTILEVER NEEDLE-BAR ON SUB-BENCH, MOFFAT TUNNEL, WEST PORTAL.

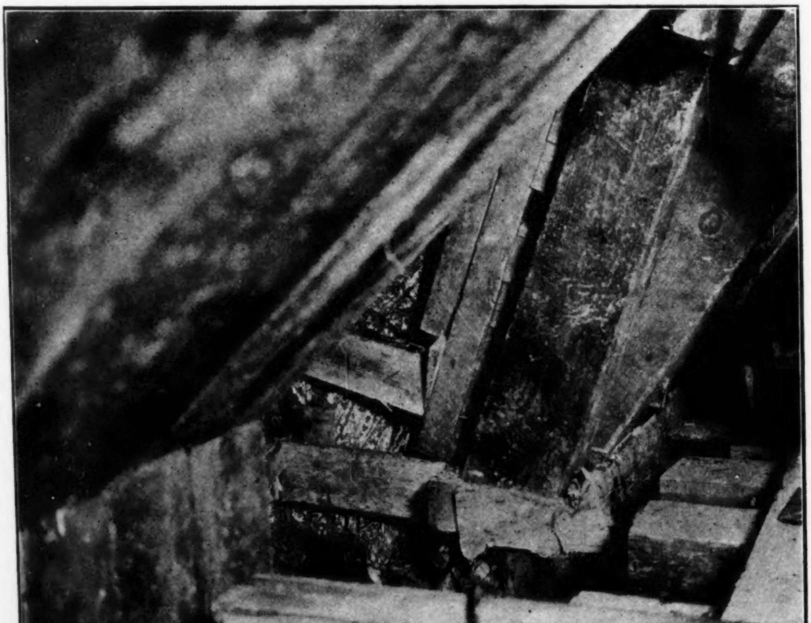

FIG. 21.—VIEW OF 12 BY 12-INCH WALL-PLATE FAILURE, MOFFAT TUNNEL, WEST PORTAL.

Timber Framing and Treating

The large amount of timbering to be done at the West Portal led to the installation of a plant for framing and notching these timbers. The dapping machine designed on the job for this purpose was a great success.

It was also considered advisable to creosote the foot-blocks. This was done by the open-tank process. Because of indications of attack by fungus growths, the advisability of treating with preservatives all the timber was seriously considered, but after a thorough study of creosoting and other processes ordinarily used it did not appear that any of them were adapted to the particular requirements of timbering for the Moffat Tunnel.

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ECONOMIC AND ENGINEERING PROBLEMS OF HIGHWAY LOCATION*

By W. W. Crosby,† M. Am. Soc. C. E.

"Economic"—like "efficient" and some other terms now in frequent use —seems to cover, more or less loosely perhaps, quite a range of meanings. Originally applying to "household" or "domestic" matters, and later pertaining to pecuniary means or concerns relating to income or expenditure, its use has grown—apparently quite properly with the development of the household or family into the community or Nation—to cover "the production, distribution and use of wealth (or resources)" and "to relate to means of living or arts by which human needs and comforts are supplied."

This interpretation is given to avoid at the outset any impression that the narrower sense of the word—such as "sparing" or "saving", not to say "mean"—might equally apply.

A few of these economic problems of highway location will occur to the engineer clearly, as for example:

(1).—What "controls"—that is, what geographical points to be selected as termini or as intermediate points—shall be determined or accepted for the highway?

(2).—By whom shall they be determined?

(3).—In their determination, what weights shall be given to such factors as:

(*a*) The character of the highway needed—that is, the probable use to be met by it after completion—and the probabilities for permanence or development of that character.

(*b*) The sources of funds available for the construction of the highway.

(*c*) The relation of the location and usefulness of the highway to the other existing or probable means of transportation serving the same areas and foci.

(4).—What controls or locations shall be selected for purposes of:

(*a*) Avoidance of traffic congestion.

(*b*) Time-saving in highway travel.

(*c*) Decentralization of population and commercial activities.

(*d*) Property development.

(*e*) Convenience and pleasure in highway use.

* Presented at the meeting of the Highway Division, New York, N. Y., January 21, 1926. The other two papers presented at this meeting by Messrs. Catchings and Breed, were published in January, 1927, *Proceedings*.

† Cons. Engr., Baltimore, Md.

(5).—Effects of existing laws and customs or lack of law and authority on the location.

(6).—Elasticity (or the lack of it) of local highway laws affecting revisions of locations.

(7).—Legal authority for the persistent protection of the public highway.

(8).—Control of the use of the highway.

(9).—Public education on this general subject.

Similarly, the engineering (technical) problems will include:

(10).—Comparisons of the results of direct or indirect alignment between controlling points, as to:

(*a*) Costs of construction.

(*b*) Costs of operation (maintenance and use).

(11).—The effects of grades on traffic costs, speeds, and operations, both for present and future motor traffic.

(12).—The effects of length on these same factors.

(13).—The determination of suitable widths to insure maximum satisfaction.

(14).—The regard to be paid to the provisions for automatic safety of traffic.

(15).—The regard to be shown for the speed (or time) factor for individual units, as well as for the average of the whole traffic.

(16).—Local physical conditions affecting construction or maintenance.

(17).—Modifications of normal construction details necessitated by the foregoing considerations.

These problems will be analyzed in the order given.

Problems (1) and (2).—Controls.—Modern road work in America has been confined mainly to the improvement of a small percentage of old roadways—roadways which for the most part had "just growed". The selection of the fraction to be improved has been made in most cases by political bodies, that is, by elected representatives of the voting population of the State, among whom the economist and the engineer have usually not been identifiable. In only one or two cases, to the writer's knowledge, has any perceptible influence from an economist or an engineer been apparent in the selection of the State highway systems.

In other words, the economic problems of controls, have been settled by politicians rather than solved by experts, and if these solutions have been really economic, it is merely a fortunate, and accidental, result. As many are beginning to realize, however, such fortuitous good results are not prevalent enough to warrant the longer neglect of this important matter. Some more scientific method must be reached, for determining between what termini main highway systems, costing millions of dollars, are to be built and through what intermediate points they may wander en route.

The railroads, as a whole, did better with their locations although some were fixed in considerable detail by political authority against the struggles of the less selfish advisers.

One reason for some of the extravagant or otherwise bad political locations is that no convincing arguments were presented against them. Usually the question of location came in the form of a choice from one or more routes between two points, the connection of which by a State highway was recognized as mandatory; and the strongest politicians won.

Until recently there has been little evidence that the economics of such a choice were considered or even presented. As a matter of fact even now there would probably be as many different styles of presentation as there were economic advisers, because of the newness and present uncertainties of the subject. As for the engineering side of the question, that, if it intruded, has usually been treated in so flattering a manner as to dismiss it completely. "The engineers will take care of that difficulty", has resulted in many a regrettable location.

In Maryland, the selection of the old routes was made under such a general authority and control as:

"Such a system of main market road connecting the various County Seats with each other and with Baltimore City as might reasonably be expected to be built with the funds provided". (The specific sum named was $5 000 000).*

The resulting State Highway System was a fairly logical selection from the existing establishment of old roads, although political influence warped the results somewhat and prevented the ultimate economy desirable. No authority was given the Highway Commission, however, to consider routes other than the existing public roads.

The propriety of the legislative action in this case was rather remarkable and seems to have offered an example to be emulated by other States, although it is not evident that many did follow that lead. To some extent the Federal Government, however, has pursued the idea.

Problem (3).—Factors Effecting Controls.—The main controls having been established, many problems as to the intermediate or minor controls of the locations arise, and with them Problem (3) involving factors governing such controls appears:

(*a*) Obviously, if a highway for through traffic is being located, indirectness must be avoided even at the expense of by-passing an intermediate community. Equally, unless the highway is to be largely for industrial traffic, directness, at the expense of avoiding points of scenic and historic interest, or of failing to serve localities through which it runs, is not warranted.

(*b*) If the sources of the funds for the highway come solely from the State Treasury or a State bond issue, for instance, the general State interest in the location must take precedence over local or individual interest. If the cost of the highway is largely met by taxation on adjacent property, the reverse is true. If a highway is being financed as an industrial and commercial proposition, its location must be regarded in a different light from another the costs of which are to be met by a general tax on the sales of gasoline.

* State of Maryland, Acts of 1908.

(c) Frequently the argument for locating a State highway to parallel closely a steam railroad is that such a location would reduce the cost because of the advantages in the supply of materials for construction. This is a pure fallacy. It would be much better to argue that State highways ought to run at right angles to railways. Of course, some conditions seem to warrant the supplementing of railway facilities by parallel highway facilities within limited areas; but considerable study and discussion are needed before principles can be well established in this matter, particularly in view of the unstable conditions now observable both as to present and future prospects in the matters of traffic developments, civic developments, and progress in mechanical equipment for movement of both persons and goods.

Problem (4).—Social Aspects.—

(a) Traffic congestion (even with the parking factor eliminated) is largely resolvable into a location problem, and would be better solved as such rather than as a police matter.

(b) The value of time in movements is becoming important and proper location is a fundamental for solving this problem. Time or speed is now a fourth dimension, added to those of length, breadth, and thickness (or grades) and applying to all solutions of highway location.

(c) Already decentralization of community life appears on the horizon, and its advent will be hastened by the daily developments in better means of communication and in facility of individual movement. Proper recognition of the facts demands that highway locations, over which much of this movement must take place, shall not oppose the other forces acting to increase the movement. The highways should really be so located as to co-operate with those forces.

(d) Contemporaneously, the opening of undeveloped property for use in connection with decentralizing movements as well as the development of this property demanded by the natural growth of population calls for suitable highway locations.

(e) As "Man does not live by bread alone", the convenience and pleasure of a people, naturally inclined to move about readily, furnished with facilities for movement, and urged on every hand to supply themselves with vehicles for individual movement at any and all times, will require suitable regard for highway locations to accommodate their needs. It is a mistake to regard highways as purely industrial means to a commercial end. The majority of highway traffic is now "pleasure traffic" according to the surveys of several States. It seems likely that it will not be otherwise for some time. One distinguished authority has written that the "saturation point" of motor vehicle ownership in America is to be calculated solely on the basis of time available for use of the car. As every one knows the curve of American leisure is upward.

Commercial highways, industrial roads, scenic highways, super-highways—all will present their own problems of location; but first—perhaps even more

so than "the poor"—there will always be "with us" the proper location of the general purpose highway.

Thus far, in most cases of organized highway improvement, a proper conception of these problems has been prevented by too great a regard for existing roads and interference with, or lack of, a view of anything outside of them; and yet it will be seen that a proper solution of the economic problems of highway location, while calling for a careful regard for existing highways, requires vision beyond these and authority to be given into competent hands to produce, through proper channels, results in roadways outside existing public rights of way.

Problem (5).—Revisions of Alignment.—Unless State highway laws permit departures from existing rights of way and even the establishment of new ones, the hope of solving some of the economic problems of highway location is lacking. If road improvement by the State is to be within the limits of existing public rights of way, nothing but mere roadway improvement can be expected and that itself must be regrettably limited.

Problem (6).—Limitation of Future Changes.—Location solutions, forced to be settled to-day for an indefinite future, may often be different from those made for the present and the immediate apparent future, when the laws give some authority for making later changes that may then prove necessary. In other words, the amount of elasticity provided for successive action affects to some extent many location solutions.

Problem (7).—Invasion of the Highway by Outside Interests.—Similarly, protective action against common tendencies toward encroachment on the highway by private or special interests has often to be considered in determining details of the location. Unless warrant exists for the highway authorities to exercise specific protective measures, these may have to be attempted through suitable location alone.

Problem (8).—Statutory Limitation of Use.—Some problems of location also have to be regarded in the light of "use control" by the highway authorities. Lacking such control, a detail of location, such as width, which again affects alignment, may require different treatment. Similarly, safety, use control, and location are intermingled. Automatic control and automatic safety are far superior to the artificially attempted variety. "Build the safety into the location" is a good slogan.

Problem (9).—Public Education.—Last in the list comes this economic problem of how best to proceed with the location to secure the necessary public education and to obtain results in the form of better roadways that will render a higher degree of public service and yield the most lasting quality of satisfaction.

Such public education is feasible. Highway authorities, particularly highway engineers, are the ones to lead in it. The Society is pre-eminently the body to start and carry on the work, first by discussion of the subject, and, later, by extending its great influence.

In 1911 the State of Pennsylvania showed some grasp of the economic problems of highway location by providing in the Acts of that year (establishing its State Highway System), the following:

"Whenever in the construction, reconstruction, maintenance, and repair of any of the State Highways it shall appear to the Commissioner that any part or portion of a State Highway, as now defined and described in this Act, is dangerous or inconvenient to the traveling public, in its present location, either by reason of grades, dangerous turns, or other local conditions; or that the expense to the Commonwealth in the construction, building, rebuilding, maintenance, and repair thereof would be too great or unreasonable, and could be materially reduced or lessened by a divergence from the road or route; the Commissioner is hereby empowered to divert the course or direction of same; and he may diverge from the line or route of same as herein described, in such direction or directions as in his discretion may seem best, in order to correct said danger or inconvenience or lessen the cost to the Commonwealth: Provided, That the said Commissioner shall first submit a plan of the proposed change to the Governor and that the same shall be approved by him."

Later, in 1921, a broader regard was evidenced by the following addition to the Highway Law:

"The State Highway Commissioner shall also have power, with the approval of the Governor, to establish the width and lines of any State Highway before or after the construction, reconstruction, or improvement of the same, not, however, exceeding the maximum width fixed by law for public roads. Whenever the State Highway Commissioner shall establish the width and lines of any such State Highway, he shall cause a description and plan thereof to be made, showing the center line of said highway and the established width thereof, and shall attach thereto his acknowledgement. Thereupon such description, plan, and acknowledgement shall be recorded in the office of the recorder of deeds of the proper county, in a separate book kept for such purpose, which shall be furnished to the recorder of deeds by the county commissioners at the expense of the county.

"No owner or occupier of lands, buildings, or improvements shall erect any building or make any improvements within the limits of any State Highway the width and lines of which have been established and recorded as provided in this section, and, if any such erection or improvement shall be made, no allowance shall be had therefor by the assessment of damages."

Under this authority the Highway Department has been operating as suggested in this paper. It has been felt obligatory to proceed conservatively and with careful regard of, and respect for, public opinion. Every effort has been made to have the public understand the reasons for disturbing established conditions or for disappointing individuals in the provision of modern roadways on the State Highway System. It is believed some success has been achieved in enlightening citizens as to better highway location.

Problem (10).—Comparisons of Alignment.—Usually between two points of control there are several possibilities for the center line of the highway. A straight line between two points may be the shortest, but Moskowsky says it is "the most tedious distance". Traffic records show it is by no means the safest. It may not be the cheapest to build, nor even the cheapest to operate. The straight-line connection may not even be practicable. Still, its directness in most cases draws the final solution toward it "as the needle to the Pole".

Problem (11).—Effects of Grades.—Usually decisions as to alignment depend largely on the grades. As a matter of fact, the effects of grades on high-

way traffic of the present and of the near future is one of the most important primary engineering problems awaiting solution.

The older factors limiting highway grades, such as the ability and endurance of draft animals, the water-proof qualities of the surface, the absence of any speed factor, and even the slipperiness of hills, have wholly or largely disappeared on main roads. Nowadays there is plenty of power for grades of twice the former maxima, there are water-proof surfaces, fair control of speed, and far less danger from slipperiness. Only for the fact of uncertainty as to what the next twenty years may hold in the way of motor vehicle development it would seem entirely practicable to adopt a general rule to the effect that the grade maxima generally accepted as proper twenty years ago could be doubled. With the necessity for limiting widths and weights of vehicles, is not the motor train a development to be expected? And what will be its abilities on grades?

Passenger cars are now able to surmount grades of 12 or 15% if not too prolonged. Trucks, however, find difficulties with such grades (or even less) particularly in descending them. Are not improvements in braking (for instance) to be expected, which will relieve that difficulty? The revolution of railroad operations through the invention of a successful air-brake, is fairly recent.

Some excellent work toward determining the effects of grades on present-day traffic has been done by T. R. Agg, M. Am. Soc. C. E., among others. Professor Agg, however, admits frankly that his opinions are more tentative than conclusive. Of course, their relation to future traffic developments is problematical.

The importance of this matter of grades can hardly be over-estimated. On its proper determination depend solutions of alignment and many other technical problems of location.

Problem (12).—Effects of Length.—The effects of length as an economic problem have been noted; but they have their engineering aspects. It will be the province of the engineer at least to adjust the relative effects of lengths and grades in first costs and to see that the consideration includes the mechanical developments to be expected.

Problem (13).—Suitable Widths.—The protection of the public interests in the roadway will throw on the engineer the duty of establishing similar widths for the location of the highway. Even the alignment will depend more or less as to its details on the width established.

The fixing of the width of the right of way for modern highways is primarily based on the necessity as to the width for the roadway and its structures. Other factors also enter, such as the opportunity for footpaths (and possibly even for horse-drawn or slow moving vehicles) public service poles and structures, clear vision for fast traffic, reasonable provision for standing vehicles, even beautification, and convenient or recreational adjuncts.

It seems in many cases that the fixation of a proper width for the public right of way is necessary to the end of protecting the roadway itself. The "set-back" of building lines beyond the right-of-way limits on public

highways outside municipalities is not, nor is it likely to become, a usual power in the hands of road authorities. Hence, the only way to accomplish the desired results seems to lie in the establishment of sufficient widths in the right of way itself.

Usually the symmetrical development of the public right of way requires that the center lines of the roadway and of the right of way coincide. There are numerous exceptions, and often quite the contrary principle must prevail. Both these two latter considerations, therefore, will affect the alignment and the location.

Problem (14).—Safety of Traffic.—There is much discussion nowadays about safety of highway traffic. In addition to having safety "built into" the location as already suggested, it frequently is practicable to adjust engineering details of alignment, widths, and grades so as to provide a minimum of danger to traffic as a whole and thereby also largely abate the need for artificial or police control of the use of the highway. Engineering may even offset to some considerable extent the dangers from the reckless individual motorist. Wherever engineering may provide automatically a decrease of danger or an increase of safety to the traffic as a whole, that solution is naturally to be preferred to the one of personal regulation.

Problem (15).—The Time or Speed Factor.—The time or speed factor in location problems must be recognized by engineers. A curve entirely practicable for traffic averaging 5 miles per hour is entirely unsuitable for 25 miles per hour. Grades that increase time schedules between termini excessively without offsetting advantages of scenery or other compensations must be abandoned for other locations more agreeable and acceptable.

Problem (16).—Effect of Physical Conditions.—Local physical conditions affecting construction or maintenance must be considered by the engineer in his determination of the location. In laying out old roads engineers often ignored the effects of periods of high water, unsuitable subsoil, snowdrifts and ice, and similar circumstances, affecting cost and even usefulness. Proper regard for such local phenomena will often introduce a weighty factor into the location.

Problem (17).—Modification of Construction Details.—A broader and more forward-looking regard for better highway location by engineers quite likely will suggest some modifications of the present standards of construction itself that have grown up more or less fortuitously and unconsciously. These technical items will include the easing of curves (affecting the alignment) and the widening of the roadway at least on curves (both as regards the effect of the speed factor); the development of the width of the locations, to provide best for its utilization with regard to convenience and safety of traffic; and all other public service interest in the highway right of way, existing or likely to prevail. Some at least of these details will affect the location.

The opportunity offered for presenting this paper has seemed to the writer best improved by thus briefly enumerating some of the problems of

highway location that have come up in his experience. Some of these many problems have not yet been satisfactorily solved except for a particular time and place. In other instances, the solutions suggested have been more expedient, perhaps, than scientific, although the effort has constantly been made to keep the science clearly in mind with a view to its best possible application.

Governor Pinchot of Pennsylvania in 1923 vetoed a bill making certain State highway locations by political selection, and Governor Smith of New York did so emphatically in 1924. Maryland, Georgia, and perhaps several other States, have avoided to some extent the political evils of selection of their State Highway Systems by legislative log-rolling. However, even where the location of the limited mileage of State highways was made by a non-partisan commission, that selection has been hampered and often rendered inferior by limiting—through lack of authority for doing otherwise—the selection to the existing old highways.

In other instances, taking Pennsylvania as illustrative of this group, the selections have been made by the Legislature itself. In Pennsylvania, although rather definite controls of the State highway routes selected were prescribed in detail, one saving clause of the State Highway Act of 1911, giving the State Highway Department power to relocate State highway routes relieved the situation extraordinarily. When to this clause were added the amendments of 1923 (as to widening of State highway rights of way) and of 1925 (legalizing the filing of a new right of way for protective purposes and future use), it will be seen that in some ways perhaps and despite any handicap of legislative establishment of controls, freedom of action in highway location in Pennsylvania has been greater than elsewhere.

In addition, the work of establishing the ultimate right of way is considered extremely important in Pennsylvania. Other States, it is to be noted, have envisaged it. Its execution, however—the securing of greater widths for the public right of way—is often an expensive matter. Always it is a difficult one. It must be approached carefully if disaster is to be avoided.

The ultimate width is inextricably involved with other details of location. Such a proposition as a legislative enactment that "all State highways shall be 100 ft. wide", even if legally sound, would probably prove enormously extravagant and eventually be unsatisfactory and disastrous unless coupled with sufficient authority to revise the alignments, that is, to relocate them, contemporaneously. Even then, with sufficient authority in proper hands for both relocation and widening, there should be some elasticity and recognition of differences in local circumstances and in needs for the widths provided, in order that the progress achieved, if slower and more gradual, may be more lasting and ultimately satisfactory both in the physical results on the highways and in the position of the highway engineer in the community.

ECONOMIC AND ENGINEERING PROBLEMS OF HIGHWAY LOCATION

Discussion*

By Messrs. L. O. Marden and Charles B. Breed.

L. O. Marden,† Assoc. M. Am. Soc. C. E. (by letter).‡—Undoubtedly, as Mr. Crosby has stated, a great many old highways were located through politics. However, several trunk lines in New England were laid out by private companies called "Turnpike Companies". Those located during the last thirty years in many cases served the towns having the strongest political representation whether or not they were on the most direct route.

At present, one of the most heavily traveled roads in Massachusetts is the "Old Newburyport Turnpike". Plans have also been made by the Massachusetts Department of Public Works, Division of Highways, for a main trunk line along certain parts of the old turnpike road between Boston and Worcester. On the other hand many of these old turnpike roads have fallen into partial disuse for various reasons:

1.—The old central village of the township through which the road passed may have lessened in importance because of a new railroad village one or two miles from it.

2.—Many old mills located in remote locations, which were near the turnpike, moved to the railroad, or never rebuilt after the buildings were destroyed by fire.

3.—Old farms have been abandoned.

4.—The grades were too steep for modern travel.

The engineers who located many of these old turnpikes should be given due credit for their monumental work in a day when the country was young and the traffic demands were less stringent.

Charles B. Breed,§ M. Am. Soc. C. E.—Mr. Crosby has not attempted to set forth any specific engineering or economic theories but has confined his paper to a more general treatment of the subject. The speaker likewise, will avoid any mathematical analysis of the relative values of length, curvature, and grades in highways. Scientific analyses of these features of location may be ever so exact in logic and mathematics and yet, due to the scarcity or

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* This discussion (of the paper by W. W. Crosby, M. Am. Soc. C. E., presented at the meeting of the Highway Division, New York, N. Y., January 21, 1926, and published in this number of *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, January 29, 1926.

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uncertainty of fundamental data, fall well short of giving results of great value.

The study of most engineering problems involves a knowledge of, and an application of, economic as well as engineering principles. It is often difficult and unwise to separate the two. The author has separated the purely economic from the purely engineering phases; but he has made clear that this separation is wholly artificial and for the purpose solely of emphasis and clarity.

Engineering facts can always be readily obtained by accurate measurements of the terrain; the possible alignment and grades submit to simple processes of surveying. The purpose of the location, however, is to furnish highway service of the character required on that particular road not only for the present but for the future. It is not merely the design of a road with flat grades and curves. The location is often spoken of as the one feature of the highway that remains for all time, or at least for several generations.

To predict the service required many years hence obviously requires superior judgment—it enters the realm of prophecy. To foretell judiciously the future service requirements of a highway obviously requires an intimate knowledge of its present requirements, the trend of development of the business and habits of the communities it will serve, the probable limitations of the vehicles used for transportation, and the effect of the engineering features, such as length, curvature, and grades on the cost of operating these transportation units. Were it possible to obtain these economic facts as accurately as it is to obtain the engineering facts the solution of the problem would be much more simple and definite. The principles of economics and of engineering are pretty well known; their application to a set of known facts is a process of accurate computation and thinking, but the facts themselves which form the premises are too frequently lamentably uncertain.

The history of railway location affords examples of this condition. Although there are many examples of poor railroad location in America the underlying fault was due in most cases to the lack of knowledge of the future sufficient to admit of a sagacious guess at the traffic to be expected. Railroads were constructed in many instances before any traffic actually existed; the service they rendered caused industry to create the traffic. As it grew in volume it became much greater than was anticipated by the most optimistic prophet. A new set of traffic facts and motive power facts then existed, which called for a radical revision in the location of parts of most of the larger railroads. Increase in traffic and development of the locomotive have made the old alignments economically unsound.

Thus it is that a poor location for one set of economic facts may be a good location for another; a poor location to-day may have been a good one 70 years ago. Engineering facts and axioms have not changed, economic principles have not changed, but the facts on which the economics was based, have been wholly altered. It is easy to criticize a location in the light of past facts, that is, judgment. Prophecy into the futúre cannot hope to reach the infallibility of judgment of the past.

In highway location the change in transportation methods from the horse to the motor vehicle has altered the whole economic program and has enor-

mously increased the amount of traffic. It is difficult to predict far into the future with relation to the ultimate limits of the motor vehicle; people are living too close to the day when no one would have dared predict what their eyes are now seeing, for them to make sanguine guesses as to the future of highway traffic. Highways must be constructed, however, and the engineer must select the route.

The proposal that the economic and engineering problems in highway location should be under the control of the engineer, strikes a sympathetic chord in any association of engineers. A poor location is often attributed to the dominance of politicians of the past. It may have been largely due to the lack of vision of engineers as to the tremendous development in transportation vehicles. Earlier settlers should not be criticized too promptly. It is not at all comfortable to feel that future generations may attribute present shortcomings in location to the dominance of the politicians of to-day, for that indicates that the engineer's influence in public affairs has been weak. Political influence does exist to-day and engineers have a very real responsibility in combatting the political selection of highway routes. That responsibility lies now, as it did then, in the apathy shown by most engineers in standing aside and permitting these influences to control.

Even where there is no doubt of its existence there are still great obstacles to successful opposition to it. The ammunition of the combat is publicity—it is a process of education of the public—the powers that control the selection of a highway route often also control the press. The knowledge of the facts and principles necessary to initiate and carry on the fight usually lies only in the mind of the engineer closely associated with the project. One naturally asks the question—would his position thereafter be a secure and happy one? It is a comforting thought for the engineer who courageously combats improper influence that public confidence in his superior ability to solve such problems is increasing because that confidence has been justified.

Errors in new location and in the improper selection of existing routes for improvement are a waste of public funds and should be avoided by removing if possible their cause, which usually is undue influence. When all improper or selfish interest is removed from effecting public improvements, presumably the millenium will have arrived. It is obviously the engineer's duty to assist in preparing the world for this important event.

Modification of the location of present highways for the most part consists of flattening curves and grades, but preserving much of the old location both in line and grades. This clinging to the old location and width of right of way may be carried too far through fear of apparently giving undue weight to a prophecy into the future. The public improvements that have been courageously developed along broad lines have seldom been regretted. It is the makeshift and niggardly policies in public affairs that have been expensive. Progressive cities have resorted to radical cures of highway transportation problems by city surgery. Why should not progressive States adopt the same policies?

Because of the ignorance of economic facts that may exist in the future it is advisable to select a location to-day which is amply wide and which with

respect to grades and curves will admit of the necessary improvement to meet almost any set of conditions that its use or service may later demand. All these improvements need not be made at once; some that are possible may never be made. That is what American railroads did; they have recently been revising grades and adopting flatter curves. If the highway location is made wide enough at curves they can be flattened when required. If the summit is located in earth rather than in ledge, its grade can be reduced by excavation and filling when the traffic justifies the expenditure. Both low and high gradient roads are the truly economical roads under certain conditions. Such a policy as this produces highways of the greatest good to the greatest number.

Any feature of location that will affect the ultimate volume of traffic should be considered, because the purpose of superior location is to stimulate traffic—to give the maximum mobility to highway transportation.

Decentralization of community life appears to be a permanent rather than a transitory movement in America. If this premise be true, then it should be given consideration in highway location. Certain it is that America is mobile. Evidence of this lies in the fact that it has one passenger automobile to every seven persons. It is also evidenced by the recent and rapid development of wayside camping grounds and cottages for overnight accommodations. Had not roads been improved, community decentralization would not have progressed.

The location of a commercial or industrial road should be safe and direct, and devoid of long, steep grades. Its selection is a problem of economic compromise or adjustment. To keep the grade below a certain maximum often involves additional length and curvature or else excessive grading costs. In few instances can the ideal be reached; and if reached to-day, who can be certain that it will apply to to-morrow's traffic?

Through routes are often diverted from the direct routes for the purpose of making use of existing roads approaching and traversing prominent intervening cities and towns. As a result of this practice, through traffic is slowed down in passing through the intermediate cities, and the more congested parts of the municipalities on this route are subjected to the hazards of this through traffic. To make such through routes more direct by by-passing the intermediate towns, and to build short, well-paved feeders from the through route to the towns for commercial purposes, would seem to be in many cases the proper way to produce the maximum highway service. Such a policy, however, is frequently met with opposition from the retail storekeepers of the towns, who desire the maximum of traffic to pass their doors. It is not the motor-truck traffic that concerns them, but the fact that on all these direct commercial routes there is a great deal of passenger and tourist travel—a fact that the locating engineer should not fail to consider.

Deviation of the main road so as to pass through intermediate towns is usually more costly to transportation, all factors considered, than the use of short branch roads. Which course to pursue depends on the relative volume of through and way traffic, for the road should best serve and stimulate traffic.

Serviceable and attractive roads cannot be obtained unless they are planned. In the esthetic phase of road betterment, the endeavor should be to conserve Nature's beauties for the convenience and enjoyment of man; it should not endeavor to create them.

Roads suitable for commercial use are usually also suitable for passenger vehicles. There is nothing criminal in making a commercial road beautiful. The roads of heavy motor-truck traffic frequently have heavy passenger traffic. This is certainly the case in New England.

The present tendency is directly toward placing the cost of highway construction as well as maintenance on the highway user. In line with this logical tax, it becomes more necessary and reasonable that the user shall be provided with safe roads that will give the maximum service per dollar of expenditure. In so far as possible, automatic safety should be put into the location. This point cannot be too forcefully emphasized.

Many States of large area and limited funds are obliged to spread expenditures to such an extent that logical development in permanent locations and with permanent safety can only be accomplished by avoiding in the selection of the yearly program those proposed improvements that may militate against a courageous and logical solution of a location or safety problem at some future time. When a State is forced to that policy, there can be little hope for rapid progress toward permanent betterment.

The location of a scenic road is often simple. Provided the road is safe, its grades may be steep and its alignment crooked. If it is a choice between a scenic route on a new right of way and an ugly road over any of the existing routes, the only way to obtain the service desired is to adopt the new location. To spend public funds to build a low-grade direct route instead of a safe winding and scenic road for pleasure driving would be the height of folly.

The approach to that phase of highway location not affected by political influence or esthetics (in this paper called the engineering phase) logically follows the method adopted by the late A. M. Wellington, M. Am. Soc. C. E.* Such a treatment involves the study of the effect of length, curvature, grades, and rise and fall on operating costs. Mr. Wellington properly separated these subjects into those which affect the number of trains required for a given traffic and those which affect only the cost of operating a given train.

The maximum grade limits the total load than can be hauled by a given locomotive. For a given traffic, therefore, it fixes the number of trains required to move it. There is nothing gained by designing locomotives with the shift-gear feature for using a low gear when climbing a grade, because the tractive effort of any locomotive at low speeds is limited by the adhesion between the wheel and the rails. Generally speaking, locomotives are designed so that at low speeds there is sufficient power to slip the driving wheels.

On maximum highway grades the coefficient of friction between the wheel and the road surface is about three times that on railroad tracks. The result is that slipping the wheels of a motor car on a firm, dry road is not at all likely on grades that would be considered reasonable on any State Highway.

* "Economic Theory of Railway Location."

The shifting from a higher to a lower gear transmits the motive power to the wheels of the motor vehicle at a slower rate, with the result that the climbing of steep highway grades is attended only with loss of time and increase in fuel consumption.

The outstanding difference between the economics of railway and of highway location lies in the fact that the low grades on a railroad are of paramount importance, whereas there is no feature in highway location—length, maximum grade, curvature, rise and fall, or width—that is so critically important to a highway as the maximum grade is to a railroad.

All the effects of these various elements in location, in operating costs on highways, fall into the second class, namely, those which affect the cost of operating the unit of transportation. In none of them is there any danger of actually stalling the vehicle or of making it physically imperative to use more vehicles.

In comparing two possible locations—one the direct and logical highway alignment from the standpoint of good economics and good engineering, the other the more popularly favored route, even if the effect of the latter on operating costs may be considerable—the degree of prominence of these unfavorable elements of greater length, steeper grades, and sharper curves, in comparison with the effect of manufactured public opinion, public apathy, personal interest, and the like, is relatively much less than in the case of a railroad. There is a grave danger, therefore, inherent in the relative strength of the forces tending to fix a highway location that undue weight of half formed or improperly formed public opinion may outweigh in the minds of legislators and public officials the sounder engineering and economical principles.

From all this one might be led to the erroneous conclusion that the only elements of highway transportation cost affected by the four different elements of location are the variable costs, such as fuel, lubrication, tires, repairs, and time—that the fixed charges, such as taxes, registration fees, interest on money invested, and depreciation, are not affected. Any element of location that increases the time consumed decreases the miles per daily use of the vehicle and therefore theoretically, and in some cases actually, reduces the number of miles per year for that vehicle. Obviously, this increases the total cost per mile for those charges which fall in the class called fixed charges. A truck which operates only 15 000 miles a year will pay exactly as much tax, registration fee, insurance, garage rent, and interest on investment as one traveling 20 000 miles.

These enormous expenditures and their tremendous effect on industry, commerce, and finance, require thorough investigating by scientific methods—more appropriately by the Federal Government—to measure the cost of length, curvature, and grades, and of rise and fall, by outdoor laboratory tests over selected routes with calibrated vehicles of like design and condition and operated by drivers of like habits.

First, establish the fundamental facts before disseminating them. Only the fundamental principles are known at present. To-day the best engineering judgment is directed toward this problem. Searching for the facts will cer-

tainly improve the judgment. The result should be the selection of location as intelligent as is the choice of type of pavement.

The U. S. Bureau of Public Roads has investigated the distribution and destructive effects of traffic loads on various pavements and subsoils; it has investigated certain structures and the properties of materials of construction. In addition, among others the Bates Road Test and the Pittsburg Road Tests were extensive and expensive studies of the behavior of pavements under intensified traffic. Further, The National Research Council is co-ordinating facts from experiments. Probably no such extensive study of the cost of different elements of location has yet been systematically made as in the investigations of pavement design, construction, and maintenance.

RECENT DEVELOPMENTS IN CONCRETE PAVEMENTS

Discussion*

By A. T. Goldbeck, Assoc. M. Am. Soc. C. E.

A. T. Goldbeck,† Assoc. M. Am. Soc. C. E. (by letter).‡—Periodically it is well to pause in any research undertaking, to look back over the established facts, note the general indications, and decide which conclusions are ready for practical application. Then the next step in advance may be taken more safely. Mr. Breed has recognized this necessity in his excellent review of the field of highway research as it applies to concrete pavement design.

Cross-Section Design.—Up to the present researches seem to have led to a theory of concrete road design which is based on a few basic facts, or at least what seem to be facts in the present state of knowledge. A high percentage of the State highway departments have accepted a design with thickened edges, and this general design seems to be well substantiated by field research and by the service results thus far obtained.

The theory on which this so widely accepted cross-section is based is elementary. It is premised in general on the following ideas:

1.—Traffic uses the entire pavement area and the pavement must be capable of supporting heavy wheel loads wherever applied.

2.—It is particularly undesirable that the pavement be cracked into slabs of small area, for rapid subsequent failure and high maintenance expense will ensue.

3.—Since a corner crack produces a small area of slab this is an undesirable condition which should be prevented.

4.—The structural strength of the concrete alone can be relied on for supporting a load applied at the corner of the slab and sub-grade support must be neglected.

5.—Concrete suffers fatigue rapidly when the bending stress in tension exceeds one-half the modulus of rupture of the concrete.

These premises, as has been pointed out in detail a number of times, lead to the design formula for edge thickness:

$$D = \sqrt{\frac{3\,P}{S}} \qquad (1)$$

in which,

D = edge thickness of pavement.
P = maximum wheel load.
S = allowable unit stress in concrete in tension which should not exceed one-half the modulus of rupture.

* This discussion (of the paper by H. Eltinge Breed, M. Am. Soc. C. E., presented at the meeting of the Highway Division, New York, N. Y., January 21, 1926, and published in January, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Director, Bureau of Eng., National Crushed Stone Assoc., Washington, D. C.

‡ Received by the Secretary, January 21, 1926.

When a center longitudinal joint is used, of such a design that the load is carried by two slab corners instead of one, Equation (1) is applicable for calculating the center thickness, *d*, as follows:

$$d = \sqrt{\frac{3\,\frac{P}{2}}{S}} = \sqrt{\frac{1}{2}}\sqrt{\frac{3\,P}{S}} = \frac{7}{10}\,D \ldots\ldots\ldots\ldots\ldots (2)$$

It is to be observed that the pavement thicknesses at edge and center, calculated in the foregoing manner, are based entirely on the necessity for preventing small slabs from forming. Moreover, the possibility of high bending stresses occurring except at the corners of pavement slabs is not accounted for in the theory given. Deformation readings taken by the U. S. Bureau of Public Roads show that the highest bending stress occurs along the edge of the slab directly under a wheel load placed at the edge. When it is remembered that this bending stress must be combined with direct tension produced by shrinkage from drying or contraction from temperature decrease, it will be seen that much higher longitudinal tensile stress will be produced in the slab than is produced by bending at the corner.

This fact, however, need not lead to the conclusion that the corner theory of design is not adequate for determining pavement thickness. It may mean, however, that with this theory a somewhat lower unit stress should be used than one-half the modulus of rupture. Another important fact is that the fatigue tests on which the selection of the unit stress has been based, were conducted on concrete in a dry condition; some tests are available showing that greater fatigue takes place in wet or damp concrete than in dry concrete. In other words, a still lower unit stress might be desirable to take account of this fact.

The net result is that, to be safe against undue cracking of concrete roads after a period of years, a unit stress of less than one-half the modulus of rupture should be used. To offset this there is the further fact that a high percentage of wheel loads are applied from 1 ft. to 2 ft. from the edge of the slab rather than at the edge, and the edge stress thus only reaches its maximum value occasionally. The question of unit stress, however, bears further watching and further study.

Reinforcing Steel.—It seems to be quite well established that the usefulness of reinforcing steel in concrete roads lies not so much in its effect on the resisting moment of the concrete, but rather in holding the concrete in close contact after a crack forms. Under certain conditions it is likewise effective in reducing the amount of cracking.* To hold the crack together, it is necessary that the steel be proportioned so that it will not be stressed beyond the yield point. A method for calculating the amount of steel necessary to accomplish this purpose has been described in detail elsewhere.†

The formula proposed for this calculation is:

$$f\,\frac{L}{2}\,W\,B = a\,S_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

* "Report of the Investigation of the Economic Value of Reinforcement in Concrete Roads," by C. A. Hogentogler, December, 1925, Annual Meeting, Highway Research Board, National Research Council.

† "The Interrelation of Longitudinal Steel and Transverse Cracks in Concrete Roads," by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., *Public Roads,* August, 1925.

in which,

L = spacing of transverse joint, in feet.
B = width of pavement, in feet.
S_1 = allowable tension in steel, in pounds per square inch.
f = coefficient of sub-grade friction.
W = weight of concrete, in pounds per square foot.
a = cross-sectional area of longitudinal steel.

Equation (3) simply states the obvious fact that the tension in the steel at a crack must equal the total sub-grade friction between that crack and a free end of the slab, such as exists at an expansion joint. It also brings out clearly that the amount of longitudinal steel which should be used depends on the spacing of expansion or construction joints.

The amount of transverse steel might be calculated by the same formula. The transverse steel should be capable of sliding the slab transversely over the sub-grade without overstressing the steel. Theoretically, this would require the largest amount of transverse steel at the center and none at the edge. Similarly, the longitudinal steel should be of greatest cross-section midway between expansion or construction joints and could taper to nothing at the joints if this were practicable. The value for f in Equation (3) might be as much as 2.0, but is generally lower than this amount.

Equations (1) and (3) form the basis for a simple theory for the design of concrete pavements.

The Sub-Grade.—Much has been written about the effect of the sub-grade. Thus far, however, the different sub-grade characteristics are not recognized in the cross-section design. It is known that the sub-grade does exert an important influence on the behavior of the road and in certain cases, as in the worst soils, a granular blanket layer seems to be of value.* Much more research is needed on the sub-grade and this should now take the form of correlation of field behavior and sub-grade characteristics. Until this is done the science of the sub-grade cannot advance properly. The laboratory procedure of soil analysis, however, has been developed to a satisfactory degree, and it is to be hoped that means will be forthcoming for carrying the field investigations to a practical conclusion. This will give a definite knowledge of how the pavement design should be changed for soils having different characteristics.

Research has developed certain fundamental facts now combined into theories which, although not entirely rational, are at least resulting in serviceable roads. Large mileages of concrete roads remain to be built in the future and a number of details of design, construction, and materials, and the economics thereof, need further investigation. Engineers cannot afford to lie back now in their research work, complacent with the thought that they have sufficient information.

* "The Present Status of Subgrade Studies," by A. C. Rose, Assoc. M. Am. Soc. C. E., *Public Roads*, September, 1925.

EVAPORATION ON UNITED STATES RECLAMATION PROJECTS

Discussion*

By Ivan E. Houk, M. Am. Soc. C. E.†

Ivan E. Houk,‡ M. Am. Soc. C. E. (by letter).§—The purpose of presenting this paper was to make the data available for general use and to elicit discussion of the subject, hoping that through such discussion additional data of similar nature would be presented. The writer feels that the purpose has been well fulfilled. Many excellent discussions and much valuable data have been forthcoming.

Both Mr. Kleinschmidt‖ and Mr. Lee¶ ask about corrections for precipitation. Rain gauges were maintained at all stations, and all records in Table 1** were corrected for precipitation; that is, they represent actual, total evaporation (observed evaporation plus precipitation). The records would not be true evaporation records if they were not so corrected. The correction for precipitation is simply one of the incidental matters to be attended to in measuring and recording evaporation data.

Mr. Kleinschmidt asks about the methods of handling pans, frequency of adding water, etc. Naturally, the methods of handling the pans varied somewhat from project to project. Observers at Class A stations, as a general rule, followed the instructions for the installation and operation of such stations issued by the U. S. Weather Bureau.†† These instructions provide that the pan shall be filled with water to within 2 in. of the top, readings made daily at about 7:00 A. M., local standard time, and additional water added, at the time of a regular observation, when the water surface has dropped 1 in. Thus, the depths of water in the pan would vary from 7 to 8 in., and the frequency of adding water would vary throughout the year, according to the rate of evaporation.

At the U. S. Department of Agriculture stations, maintained on the Experiment Farms by the Irrigation Officials of the Bureau of Public Roads, where the 24-in. by 6-ft. circular pans are used, the water surface levels are kept from 2 to 4 in. below the rims, so that the depths in the pans vary from

* Discussion of the paper by Ivan E. Houk, M. Am. Soc. C. E., continued from October, 1926, *Proceedings*.

† Author's closure.

‡ Engr., U. S. Bureau of Reclamation, Denver, Colo.

§ Received by the Secretary, December 28, 1926.

‖ *Proceedings*, Am. Soc. C. E., April, 1926, Papers and Discussions, p. 737.

¶ *Loc. cit.*, August, 1926, Papers and Discussions, p. 1189.

** *Loc. cit.*, January, 1926, Papers and Discussions, pp. 44 *et seq.*

†† *Circular L*, Instrument Div., U. S. Weather Bureau.

20 to 22 in. Apparently, the intention is to keep the water surface at the same elevation as the ground surface, since the pans are set in the ground to the same depths. At the other stations water surface levels are kept from 1½ in. to 6 in. below the rims. Usually it is necessary to maintain lower surfaces at the floating-pan stations than at the land-pan stations, to avoid losing water by splashing during windy weather. For instance, at the Nelson Reservoir on the Milk River Project of Montana, No. 29* in Table 1, the engineer in charge first tried submerging the pan to within about 2 in. of the rim, and keeping the water within the pan at the same elevation as the water outside. However, he soon found that greater free-board was necessary. Consequently, he changed the depth of submergence, and the depth of water inside, to 6 in., leaving a free-board of 4 in. This helped materially in reducing sloshing troubles, but did not entirely eliminate them.

The majority of the stations are provided with stilling wells and accurate hook-gauges. At such places the depths of evaporation are measured by direct readings of changes in the surface level. However, in a few cases the pans are provided with needle-points, set at fixed levels, and the evaporation losses determined by adding measured quantities of water until the water surfaces rise to the fixed levels.

Mr. Lee mentions the lack of necessary descriptive data for some of the stations in Table 1. At the time Table 1 was prepared all the available information was included. Letters had been written to the Project Superintendents, asking for complete installation details and other pertinent data. However, in some cases the information could not be furnished owing to the stations having been discontinued, the observers having moved away, the records having been misplaced, or some other legitimate reason. Since that time some of the missing data have been located and consequently the following additional notes can be presented.

At the Spanish Fork Station on the Strawberry Valley Project of Utah (No. 8† in Table 1), the evaporation pan was 17 in. deep by 3 ft. square, and was set in the ground 15 in.

At the Cold Springs floating-pan station on the Umatilla Project of Oregon, for the years 1909 to 1913 (No. 21‡ in Table 1), a square pan was located in the center of a raft and was partly submerged. The raft was placed inside a hollow square made of floating timbers, to which it was tied loosely at the corners; and the whole arrangement was anchored.

At the Nelson Reservoir floating-pan station on the Milk River Project of Montana (No. 29* of Table 1), the depth of submergence, and depth of water inside, was 6 in., as noted previously. At this station it was found advantageous to mount the pan on a square raft with a triangular protuberance, and to hold the raft by a cable about 200 ft. long, attached to a single anchor, thus allowing the raft to swing around keeping its nose continually against the wind.

* *Proceedings*, Am. Soc. C. E., January, 1926, Papers and Discussions, p. 50.
† *Loc. cit.*, p. 45.
‡ *Loc. cit.*, p. 48.

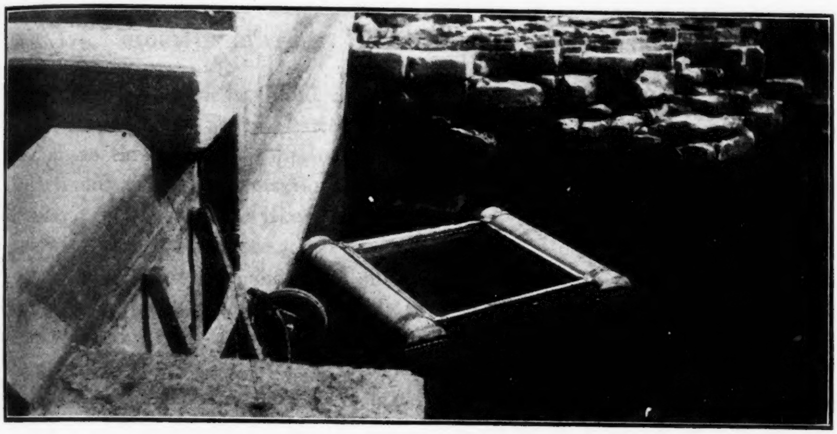

FIG. 22.—FLOATING EVAPORATION PAN AT AVALON RESERVOIR, CARLSBAD PROJECT, RECORD 1, TABLE 1.

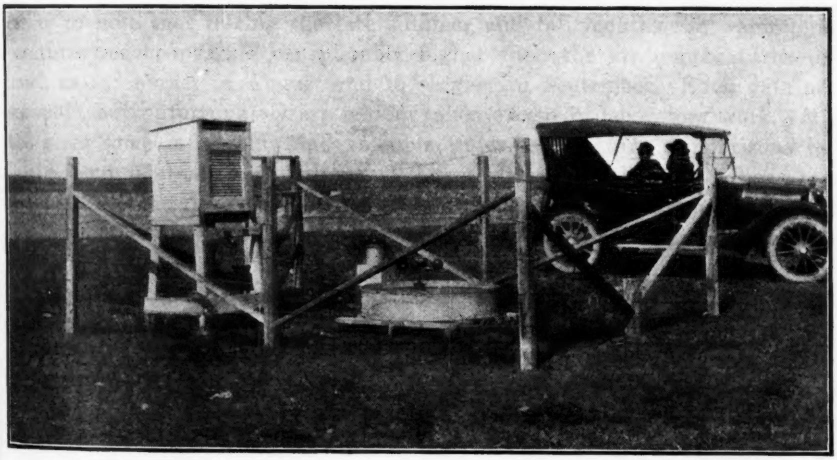

FIG. 23.—LAND EVAPORATION PAN AT NELSON RESERVOIR, MILK RIVER PROJECT, RECORD 29, TABLE 1.

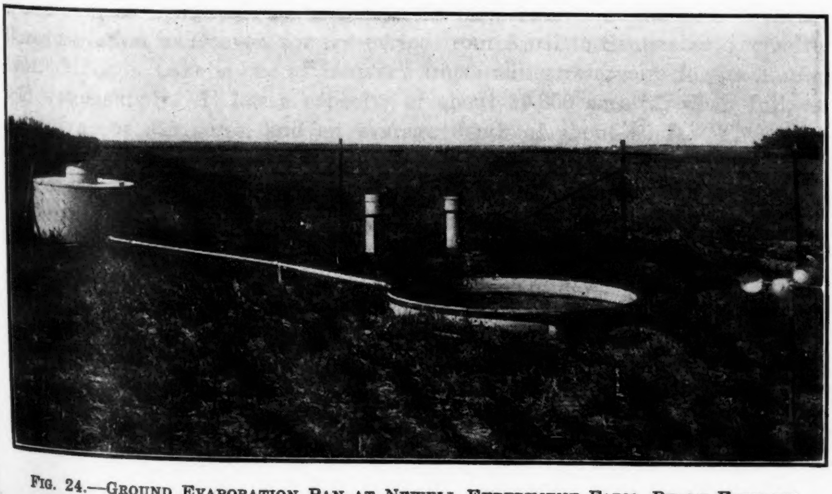

FIG. 24.—GROUND EVAPORATION PAN AT NEWELL EXPERIMENT FARM, BELLE FOURCHE PROJECT, RECORD 47, TABLE 1.

No information is at hand regarding the setting of the pan at the Hermiston Experiment Farm Station on the Umatilla Project of Oregon. However, since this station is maintained by the Bureau of Public Roads, U. S. Department of Agriculture, and since the pan is its standard 24-in. by 6-ft. circular type, it is quite likely that it is set in the ground 21 in., in accordance with the usual custom.

Figs. 22, 23, and 24 show the submergence, mounting, and exposure conditions at some of the stations. It was not considered necessary to give complete mounting details for all the Class A installations, inasmuch as the specified mounting for such stations is clearly shown in *Circular L* of the Instrument Division, U. S. Weather Bureau, previously mentioned.

Table 14,* presented by Mr. Torpen, gives valuable evaporation and related meteorological data for Lake Cushman, near Tacoma, Wash. One of the unusual features of the table is the column of water temperatures. It is interesting to note that during the late summer and fall months the water temperatures reached values considerably higher than the air temperatures—as much as 12° higher in August and 10° higher in September. Such data are desirable, but unfortunately are seldom taken except in laboratory work. Mr. Lee gives some interesting and valuable water temperature comparisons for floating and land-pan installations at Lake Almanor, California (Figs. 19,† 20,‡ and 21§), but does not include air temperatures. His assumptions regarding the temperature data in Table 1, are correct. The data are all air temperatures, based on maximum and minimum readings of instruments housed in standard U. S. Weather Bureau thermometer shelters. Occasional water temperature readings have been taken at some of the stations, but the records were not comprehensive enough to warrant their inclusion in the table; although it may be interesting to note, in connection with Mr. Lee's Fig. 20, that observations at the Lake Tahoe floating-pan station, Newlands Project, showed water temperatures in the pan from 1° to 4° colder than the river water early in the morning, and from 1° to 4° warmer in the evening.

Fig. 25 shows temperatures of the water surface in Marston Lake, about ten miles from Denver, Colo., and mean air temperatures at the U. S. Weather Bureau Station in Denver, for the period from April to September, inclusive, 1924. Marston Lake is one of Denver's impounding reservoirs for its municipal water supply. It has a capacity of about 19 800 acre-ft., when full, an area of about 650 acres, and an average depth of about 30 ft. The water temperatures were secured through the courtesy of D. D. Gross, Chief Engineer of the Denver Board of Water Commissioners. Data on the exact times of observation are not at hand but the temperatures were probably measured during the warmer part of the day. It will be noticed from Fig. 25 that the water surface temperatures follow the air temperatures very closely. Individual readings vary from about 10° below the air temperatures to about 10°

* *Proceedings*, Am. Soc. C. E., April, 1926, Papers and Discussions, p. 751.

† *Loc. cit.*, August, 1926, Papers and Discussions, p. 1198.

‡ *Loc. cit.*, p. 1199.

§ *Loc. cit.*, p. 1201.

above, while the monthly mean water temperatures do not differ from the mean air temperatures by more than 1 or 2 degrees.

Fig. 26 compares air and water temperatures at the Los Griegos evaporation station near Albuquerque, N. Mex., for the period from September 14 to October 23, 1926. This station was established in September, 1926, for the purpose of measuring evaporation from soil and water surfaces in the Middle Rio Grande Valley, the data being needed for studies of water supply available for irrigation. The station is equipped with a Class A evaporation pan, a ground pan for measuring water surface evaporation, five soil evaporation tanks, maximum and minimum thermometers, a rain gauge, and an anemometer. The ground pan is 4 ft. in diameter, 2 ft. deep, and is set in the ground with the rim projecting about 2 in. The depth to ground-water outside the pans varied from 1.1 to 1.7 ft. during the months of September and October. Observations were made at about 5:00 P. M. daily; consequently, the observed water temperatures approximate the maximum daily values rather than the mean daily values. Temperatures of the water in the ground pan were slightly higher than the ground-water temperatures, but lower than the Class A pan temperatures, as would be expected. Class A pan temperatures were nearly as high as the maximum air temperatures.

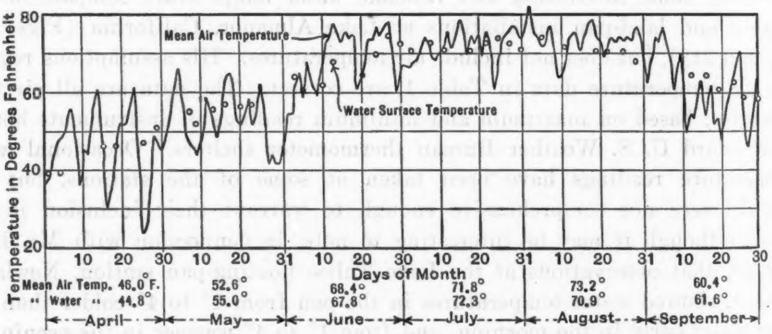

FIG. 25.—TEMPERATURES OF WATER SURFACE IN MARSTON LAKE, NEAR DENVER, COLO., AND MEAN AIR TEMPERATURES AT DENVER, APRIL TO SEPTEMBER, INCLUSIVE, 1924.

The necessity for observing water temperatures in connection with evaporation measurements depends on the type of station and the purpose for which it is maintained. At Class A land-pan stations, as well as at land-pan stations where shallow pans are set in the ground, it is doubtful whether the mean temperature of the water in the pan varies appreciably from the mean air temperature, at least as far as monthly averages are concerned. Consequently, it is doubtful whether water temperature measurements at such stations are necessary, although, of course, the data would furnish interesting information. It is desirable to observe water temperatures periodically at all floating-pan stations, timing the observations in such a way as to obtain mean daily values. However, the measurement of maximum and minimum air temperatures should not be dispensed with at floating-pan stations; both air and water temperatures should be observed. If the floating or land-pan station is installed primarily as an experimental plant, for the purpose of

investigating evaporation laws, it is especially important to obtain accurate data on both air and water temperatures, as well as on all other affecting meteorological phenomena.

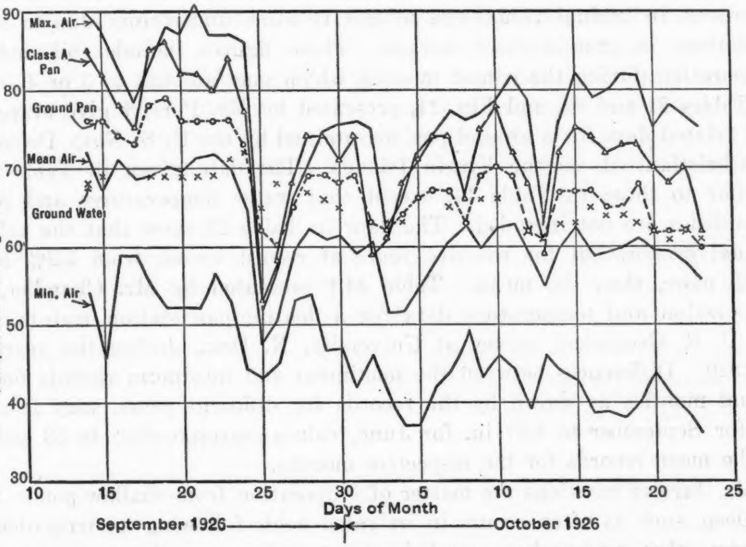

FIG. 26.—COMPARISON OF AIR AND WATER TEMPERATURES AT LOS GRIEGOS EVAPORATION STATION, NEAR ALBUQUERQUE, N. MEX.

Mr. Lee furnishes a very interesting discussion of the effect of specific gravity on the rate of evaporation. His Fig. 18* shows conclusively that the presence of alkaline salts in appreciable proportions results in a marked decrease in evaporation. The writer does not recall ever having seen this phase of evaporation mentioned, although it is a matter which doubtless should be considered in studying evaporation from alkali swamp lands such as exist in many parts of Western United States. Mr. Lee also includes a very interesting treatment of evaporation from capillary films, evaporation from moist crystal deposits, consumptive use of irrigation water, plant transpiration, and soil evaporation accompanying growth of vegetation. These matters are really phases of evaporation, using this term in its broadest sense; just as the evaporation from water surfaces is one phase of the general subject of evaporation. The U. S. Bureau of Reclamation has been collecting data on some of these matters for several years, particularly on the consumptive use of irrigation water; but the information has never been correlated in one report, in shape suitable for publication, and consequently cannot be included in this discussion.

It may be interesting to note that the difference between rainfall and run-off in some of the agricultural valleys of Eastern United States can be taken as an indication of the consumptive use of water. For instance, in the Miami Valley above Dayton, Ohio, an area including approximately

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1192.

2 500 sq. miles, practically all of which is farm land producing paying crops each year, the average difference between annual rainfall and run-off for the 25-year period from 1894 to 1918, amounted to 25.20 in., or 2.10 ft.* The twenty-five individual values varied from 1.54 ft. in 1913 to 3.17 ft. in 1896. However, individual values are subject to some uncertainty due to possible variations in ground-water storage. These figures include soil and snow evaporation during the winter months, which may amount to 3 or 4 in.

Tables 21 and 22, and Fig. 11, presented by Mr. Perry,† give evaporation and related data for a ground pan maintained by the U. S. Navy Department at Christiansted, on the Virgin Islands. The data given in Table 21 are similar to those in Table 14, except that water temperatures and relative humidities are not included. The data in Table 22 show that the values of annual evaporation for the five years of record varied from 5.2% less, to 7.7% more, than the mean. Table 44,‡ presented by Mr. Chandler, gives evaporation and temperature data for a floating-pan station maintained by the U. S. Geological survey at University, N. Dak., during the years 1905 to 1920. Differences between the maximum and minimum records for individual months, as shown by the records for different years, vary from 1.92 in. for September to 3.87 in. for June, values corresponding to 53 and 78% of the mean records for the respective months.

Mr. Jarvis§ mentions the matter of evaporation from shallow ponds, 2 or 3 in. deep, such as often occurs in irrigated fields following an irrigation, and suggests that under those conditions rates of evaporation may exceed the rates measured at either land or floating pans. The writer believes this to be true. However, he does not recall ever hearing of any evaporation measurements made under an environment approximating those conditions, although such experiments would be valuable and might easily be made at most experiment stations. It is to be hoped that practical investigations of this nature will be made some time, and that they will consider the effect of plant shade and wind protection in reducing the evaporation rate, as well as the maximum evaporation rates experienced in open ponds.

Both Mr. Follansbee‖ and Mr. Meeker¶ discuss the matter of converting pan-evaporation records into equivalent reservoir evaporation. They contribute tables of evaporation data, converted to reservoir equivalents on the basis of the coefficients determined by Sleight, at Denver, in 1916,** including in their tables records at some stations in Western United States not contained in Table 1. The writer has no criticisms to make regarding Mr. Sleight's excellent experimental work. However, in view of the fact that his coefficients have never been adequately confirmed by independent observations in other localities it seemed better to present the original records in Table 1, rather than converted values, so that engineers using the data can apply whatever

* "Rainfall and Run-Off in the Miami Valley," by Ivan E. Houk, M. Am. Soc. C. E., Miami Conservancy Dist. Technical Repts., Pt. VIII, Dayton, Ohio, 1921.

† *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, pp. 1175-1176.

‡ *Loc. cit.*, October, 1926, Papers and Discussions, p. 1692.

§ *Loc. cit.*, May, 1926, Papers and Discussions, p. 1012.

‖ *Loc. cit.*, April, 1926, Papers and Discussions, p. 737.

¶ *Loc. cit.*, p. 743.

** *Journal of Agricultural Research*, Vol. X, No. 5, July 30, 1917.

coefficients they consider most applicable. Incidently, it might be mentioned that the U. S. Bureau of Reclamation is co-operating with the U. S. Bureau of Public Roads in measuring the actual evaporation at the East Park Reservoir on the Orland Reclamation Project of Northern California, and that floating and land-pan records are being obtained at the same time, so that the ratios of reservoir evaporation to pan evaporation can be determined. While conclusive results are not available at this time, preliminary measurements seem to indicate that the ratio of reservoir evaporation to Class A pan evaporation at that particular reservoir is less than 0.66. The geological conditions are such that the quantity of water lost by seepage is believed to be negligible.

TABLE 45.—COMPARISON OF EVAPORATION COEFFICIENTS BASED ON DIFFERENT LENGTHS OF RECORD AT THE U. S. BUREAU OF PUBLIC ROADS EXPERIMENT STATION, IN DENVER, COLO.

Tank No.	Description.	Evaporation in Percentage of Evaporation from Ground Tank 12 Feet in Diameter by 3 Feet Deep. Records for 1915–1916.*	Records for 1915–1917.	Differences
1......	Ground tank, 1 ft. in diameter by 3 ft. deep..........	154.9	158.9	4.0
2......	Ground tank, 2 ft. in diameter by 3 ft. deep..........	128.7	128.4	0.3
4......	Ground tank, 3.31 ft. in diameter by 3 ft. deep.......	120.9	120.2	0.7
5......	Ground tank, 6 ft. in diameter by 3 ft. deep..........	109.6	108.9	0.7
8......	Land pan, Class A, 4 ft. in diameter by 10 in. deep	150.0	142.1	7.9
9......	Floating pan, 3 ft. square by 1.5 ft. deep...........	108.9	112.5	3.6

* From *Journal of Agricultural Research*, Vol. X, No. 5, July 30, 1917.

Mr. Sleight* furnishes a very interesting account of the experimental evaporation work carried on by the U. S. Bureau of Public Roads, at Denver, during the period from November, 1915, to September, 1917, including tabulated summaries of data for the entire period. It appears that the coefficients based on the complete records differ slightly from those based on the 1916 records.† In order to bring out this point Mr. Sleight has furnished the writer with the data in Table 45 and asked him to present it in his closing discussion. It will be noticed that the difference is greatest in the case of the U. S. Weather Bureau Class A pan; that it is appreciable in the case of Tanks Nos. 1 and 9 (the ground tank 1 ft. in diameter by 3 ft. deep and the floating pan 3 ft. square by 1½ ft. deep); and that it is insignificant in the case of the ground tanks, Nos. 2, 4, and 5 (tanks, 3 ft. deep, having diameters of 2, 3.31, and 6 ft., respectively). The data in Table 45 indicate the desirability of continuing evaporation experiments through a number of years rather than discontinuing them after records for one year or two years are obtained.

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1162.

† "Evaporation from the Surfaces of Water and River-Bed Materials," by R. B. Sleight, Assoc. M. Am. Soc. C. E., *Journal of Agricultural Research*, Vol. X, No. 5, July 30, 1917.

Regarding the ratios of ground-tank evaporation to Class A pan evaporation at Jackson Lake and Denver, discussed by Mr. Sleight, it is likely that the ground-pan evaporation at Jackson Lake was relatively lower than that at Denver due to a higher ground-water level. C. C. Elder, Assoc. M. Am. Soc. C. E., who installed the Jackson Lake ground pan and took the records during the years 1922 and 1923, states that the pan was in a swampy location about 6 ft. from a small drain channel, in which the water was always rather cold, probably never warmer than 50° Fahr. The pan was set in the ground to half its depth, and earth banked around the outside as high as the rim. It was filled with the comparatively cold drainage water. Since the ground-water was very close to the surface, sometimes at the surface, it must have had an appreciable cooling effect on the water within the pan. Somewhat similar conditions exist at the Los Griegos Evaporation Station, near Albuquerque, previously mentioned. Ratios of ground-pan evaporation to Class A pan evaporation at Los Griegos for the months of September, October, and November, 1926, were 79.0, 75.3, and 73.2%, respectively, averaging 75.8 per cent. During these months the depth to ground-water varied from about 1 ft. to 2 ft., being higher than the bottom of the pan the greater part of the time.

Mr. O'Neil* furnishes a very interesting account of his evaporation investigations at the Lake Newell and Brooks Reservoirs, Alberta, Canada, including valuable tables of evaporation data, ratios of floating to land-pan evaporation, reservoir losses, etc. The ratios of floating to land-pan evaporation, given in Table 31,† were determined at Brooks during the six years from 1919 to 1924. While their average value is 1.28, it is interesting to point out that the individual values vary from 1.08 to 1.40. This again indicates the desirability of continuing evaporation experiments through a number of years. Inasmuch as the land pan at Brooks was set in the ground it is not surprising that the differences between the land pan and floating-pan evaporation should be less than the differences found at locations where Class A land-pan installations have been used. However, it is rather surprising that the floating-pan evaporation should be consistently higher than the land-pan evaporation.

The writer is inclined to believe that the actual winter losses, given in Table 30,‡ may be fully accounted for by evaporation. They vary from 0.04 to 0.20 ft., or from 0.48 to 2.40 in., averaging 1.32 in. Such a total loss corresponds to a mean daily loss of 0.0147 in. for the 90-day period from December 1 to February 28, an amount which is not at all inconsistent with available data on evaporation from snow and ice surfaces. By referring to Table 4§ it will be seen that the ice evaporation measurements on the Milk River Project in 1922 and 1923, showed average daily rates of 0.013, 0.010, and 0.015 in., respectively, for the months of December, January, and February. The average temperatures on the Milk River Project during January and February were lower than those for corresponding months at Lake Newell,

* *Proceedings*, Am. Soc. C. E. August, 1926, Papers and Discussions, p. 1202.

† *Loc. cit.*, p. 1207.

‡ *Loc. cit.*, p. 1206.

§ *Loc. cit.*, January, 1926, Papers and Discussions, p. 61.

given in Table 33.* It might also be mentioned that observations in the Miami Valley of Southwestern Ohio, during the severe winter of 1917 and 1918, showed an average rate of snow evaporation of 0.023 in. per day for the period from December 3, 1917, to February 11, 1918.† During this period the average temperature was 19° Fahr., the mean wind velocity, 11.5 miles per hour, and the mean relative humidity, 83 per cent. R. E. Horton, M. Am. Soc. C. E., obtained an average rate of snow evaporation of 0.028 in. per day for the nine days from December 26, 1913, to January 4, 1914, at Albany, N. Y.,‡ when the mean maximum temperature was 26.6° Fahr.

Table 46 contains an abstract of snow and ice evaporation measurements made on the Milk River Project, of Northeastern Montana, since January, 1924. It is a continuation of Table 4. Measurements used in compiling Table 46 were made at Malta instead of Saco, the apparatus having been installed at Malta in November, 1923. Since that time evaporation readings have been taken through the summer as well as through the winter.

TABLE 46.—ICE AND SNOW EVAPORATION ON THE MILK RIVER PROJECT, MONTANA, DURING WINTER MONTHS, FROM 1924 TO 1926.*

Period.	Number of days observed.	Total evaporation, in inches.	Average rate, in inches per day.	Equivalent, in inches per month.	Mean temperature, in degrees Fahrenheit.	Surface conditions.
1924:						
Jan. 1-31.	31	0.56	0.018	0.56	4	Snow and ice.
Feb. 14-26.	13	0.36	0.028	0.78	13	Ice.
Mar. 1-31.	31	1.43	0.046	1.43	26	Mostly ice. water around edges at times.
Dec. 15-31.	17	0.37	0.028	0.67	—8	Solid ice with 2 to 3 in. of snow on top.
1925:						
Jan. 1-17.	17	0.17	0.010	0.31	7	Solid ice with 1 to 3 in. of snow on top.
Jan. 18-23.	6	0.32	0.053	1.65	33	Ice with 1 to 3 in. of water around edge.
Jan. 24-29.	6	0.11	0.018	0.57	13	Solid ice, bare.
Feb. 11-28.	18	0.61	0.034	0.95	14	Solid ice with 1 to 2 in. of snow on top at times.
Mar. 9-17.	9	0.29	0.032	1.00	12	Solid ice with 2 to 4 in. of snow on top at times.
Mar. 18-22.	5	0.21	0.042	1.30	31	Ice with ½ in. to 2 in. of water around edge.
Nov. 22-30.	9	0.67	0.074	2.23	25	Ice with 1 to 2 in. of water around edge at times
1926:						
Jan. 1- 9.	9	0.84	0.093	2.89	20	Solid ice with 1 to 4 in. of snow on top.
Jan. 10-17.	8	0.72	0.090	2.79	30	Ice with ½ in. of water around edge.
Jan. 18-31.	14	0.52	0.037	1.15	19	Bare ice.
Feb. 13-21.	9	0.15	0.017	0.47	25	Ice.
Mar. 7-14.	8	0.31	0.039	1.20	34	Ice with 1 to 2 in. of water around edge.

* See Table 4 (*Proceedings*, Am. Soc. C. E., January, 1926, Papers and Discussions, p. 61), for earlier data.

G. E. Stratton, M. Am. Soc. C. E., Superintendent of the Milk River Project at the time the measurements were made, advises that in the winter of 1923 and 1924, and since about March 1, 1926, a semi-circular sheet of flume metal was placed around the small pan so as to protect its sides from

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1208.

† "Rainfall and Run-Off in the Miami Valley," by Ivan E. Houk, M. Am. Soc. C. E., Miami Conservancy Dist., Technical Repts., Pt. VIII, Dayton, Ohio, 1921, p. 65.

‡ "Evaporation from Snow and Errors of Rain Gage when Used to Catch Snowfall," by R. E. Horton, M. Am. Soc. C. E., *Monthly Weather Review*, February, 1914, p. 99.

the direct action of the sun during the greater part of the day. The water surface was kept about 8 in. below the rim during the winter months, so as to reduce freezing and bursting troubles as well as to provide space to catch the snowfall. During the summer months the water surface was kept about 2 in. below the rim.

The data in Table 46 show conclusively that evaporation continues at appreciable rates during the winter months, even when the temperatures are below zero. Daily rates vary from about 0.010 to about 0.093 in., for the periods included. While the values are somewhat erratic, as a general rule the higher rates correspond to the higher temperatures.

Table 47 compares the evaporation measured in the small pan at Malta, with that measured in a Class A pan mounted near-by. The interesting point brought out by Table 47 is the effect of shading the sides of the small pan. During the months of August and September, 1924 and 1925, when the sides of the small pan were not shaded, the ratios of Class A pan evaporation to small pan evaporation, varied from 53.3 to 64.5%, averaging 59.3 per cent. During the summer of 1926, when the small pan was shaded, the corresponding ratios varied from 79.4 to 92.5%, averaging 85.3 per cent.

TABLE 47.—RATIOS OF CLASS A PAN EVAPORATION TO SMALL PAN EVAPORATION AT MALTA, MONTANA, 1924 TO 1926.

Month.	Year.	Evaporation, in Inches.		Ratio: Class A pan to small pan, in percentage.	Average ratios, in percentage.	Remarks.
		Class A pan.	Small pan.			
August	1924	6.85	10.62	64.5	....	Sides of small pan not shaded.
September	1924	5.61	9.18	61.1	....	
August	1925	8.70	14.94	58.2	....	
September	1925	4.43	8.31	53.3	59.3	
May	1926	7.54	8.48	88.9	....	Sides of small pan shaded by sheet of metal flume.
June	1926	8.36	10.52	79.4	....	
July	1926	10.21	12.71	80.3	....	
August	1926	7.84	8.47	92.5	85.3	

Professor Harding* and Mr. Lippincott† discuss the matter of actual reservoir evaporation and its relation to pan records, submitting data secured on California lakes as a basis for their discussions. Professor Harding submits data for Tulare Lake, Buena Vista Lake, and Lake Elsinore; while Mr. Lippincott submits data for these three and several additional reservoirs, summarizing his observations in Table 43.‡ In some recent correspondence Professor Harding the writer was advised that the evaporation data in Tables 23,§ 25, and 26‖ have been corrected for precipitation, that no appreciable quantity of aquatic growth was present in any of the three lakes, and that the land pan at Buena Vista Lake was set in the ground to within 3 in. of the rim, the

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1185.

† *Loc. cit.*, p. 1210.

‡ *Loc. cit.*, p. 1221.

§ *Loc. cit.*, p. 1186.

‖ *Loc. cit.*, p. 1188.

water surface in the pan being kept 3 in. below the top. The slight differences in evaporation data for Tulare Lake, as given in Tables 23 and 36,* are probably due to slightly different methods in working up the basic data. For instance, one man may have used some records which the other had discarded.

Mr. Parshall† contributes an interesting account of the comprehensive evaporation measurements carried on by the Bureau of Public Roads, U. S. Department of Agriculture, at its laboratory in Fort Collins, Colo. Apparently, the measurements were made with unusual care and accuracy. The ingeniously devised optical evaporimeter is especially worthy of mention.

A rather hasty study of Figs. 16‡ and 17§ seems to disclose an unusual confirmation of the Dalton theory. In both cases the observed evaporation is seen to vary directly with the quantity, $(E_s - E_d)$, which represents the difference between the vapor pressure corresponding to the temperature of the water surface and the actual vapor pressure in the air immediately above. Mr. Parshall calls attention to this in his statement† that,

"During this period the temperatures of air and water were decreasing, as was also the rate of evaporation, which is contrary to the findings inside the laboratory under still air conditions; however, in both cases the rate of loss is directly proportional to the difference in vapor pressures."

Mr. Parshall gives two formulas for computing evaporation; one for still air conditions, such as existed in the laboratory when the observations platted in Fig. 16 were taken, the other for windy conditions, such as existed outside when the measurements recorded in Fig. 17 were made. It is understood that both equations are tentative since he states that "inasmuch as the study of this subject has not been fully developed, certain minor alterations or combinations may later permit closer agreement with the observed rate of loss." In studying the formulas one notices that in both cases the computed evaporation varies with the quantity, $(E_s - E_d)$, that in the second case a wind factor is introduced, and that in the first case a function of the quantity, $(T_a - T_s)$, the temperature of the air 1 in. above the water surface minus the temperature of the water surface, is introduced.

There is a certain theoretical basis for the variation of evaporation with the quantity, $(E_s - E_d)$, aside from the observational basis shown by Figs. 16 and 17. The introduction of a wind factor of some kind to take care of the wind effect is logical. However, the writer would be inclined to question the introduction of the function of $(T_a - T_s)$, even for purely local laboratory conditions. It would seem that the effect of the difference in temperature of the air and water could be taken care of by some function of the difference in vapor pressures, since the latter quantity is dependent on the air and water temperatures and the relative humidity. Moreover, it would hardly seem probable that the law of evaporation inside the laboratory differs from that outside. It would seem more logical to have one formula for both

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1212.
† *Loc. cit.*, p. 1177.
‡ *Loc. cit.*, p. 1183.
§ *Loc. cit.*, p. 1184.

conditions, and to obtain the evaporation rate for still air by the simple expedient of making the wind factor equal zero. If this is done in the second formula the equation becomes $E = 0.44\ (E_s - E_d)$, while if T_a is assumed equal to T_s the first formula becomes $E = 0.166\ (E_s - E_d)$. Values of evaporation computed by the latter equation would be only about 38% as large as those computed by the former, for the same meteorological conditions. However, the matter is only of academic interest to engineers, inasmuch as their problems do not necessitate the calculation of evaporation within a laboratory. They are concerned solely with the second formula, the equation for outdoor evaporation.

In the second formula the wind correction factor, $(0.44 + 0.118\ W)$, is a straight-line function of the wind velocity. In other words, its value increases directly with the wind velocity as long as that velocity increases, thus indicating an increase in the rate of evaporation with an increase in wind velocity regardless of the amount of the wind. While such a correction may be satisfactory for the range in wind velocity of from 0 to 12 miles per hour experienced at Fort Collins, the writer would not expect it to hold for indefinite increases in velocity. As far as a general evaporation formula is concerned he would be inclined to favor some wind correction term along the lines recommended by Mr. R. E. Horton.*

Practising engineers must depend on experimental laboratory work, of such nature as that carried on at Fort Collins, to determine the physical laws governing evaporation phenomena. It is to be hoped that detailed descriptions of the Fort Collins work, together with complete data, will be made available in the near future. What engineers need most at the present time, as regards evaporation, is a general formula in which they can insert mean values of temperature, wind velocity, and relative humidity, observed at U. S. Weather Bureau stations, and calculate reliable values of reservoir evaporation for exposed areas of different sizes.

* *Engineering News-Record*, April 26, 1917.

STRESSES IN THICK ARCHES OF DAMS

Discussion*

BY B. F. JAKOBSEN, M. AM. SOC. C. E.†

B. F. JAKOBSEN,‡ M. AM. SOC. C. E. (by letter).§—In Article XIX‖ of the paper, Professor Résal's formulas and the maximum stress given by Equation (83)¶ were discussed. The writer should have stated explicitly that this equation was derived for a loading parallel to the center line at the crown and not for radial loading.** Therefore, the abutments in Fig. 23†† should have been shown parallel, with L as the distance between them. Résal also assumes (see Fig. 24††), that the length of the parabolic arc, ds, is approximately equal to its projection, dy, on the Y-axis, and this is true only for a flat arc. The secondary parabolic arch for a short thick arch is far from being flat and for this condition the Résal formulas become unavailable.

In designing the Pacoima Dam it became clear that the secondary arch plays a considerable rôle in determining the maximum stress. The subject, therefore, will be further discussed.

In order to avoid undue complications and generalities, it is expedient to consider a specific case. Consider, therefore, the arch at Elevation 1 700 in the Pacoima Dam, shown in Fig. 51 with the irregular abutments, as they exist, except for the stepping of the abutment. The secondary arch as determined from the writer's formulas and from Fig. 21‡‡ is also shown. The dimensions of this secondary arch are, $r_e = 60$ ft.; $t = 30$ ft.; $\frac{t}{r_m} = 0.667$; and $2\,\phi_0 = 152$ degrees. Résal's secondary arch, determined from Equation (85),¶ is only 24 ft. thick.

There are two objections to the secondary arch. The first is that the actual load does not coincide with the assumed load. The actual load is normal to the primary arch, whereas the load is assumed to be normal to the secondary arch. Considerable calculation has convinced the writer that this objection is not of much importance, especially since the actual loading is not known, but is taken as equal to the water pressure, which is, of course, not exact.

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* Discussion of the paper by B. F. Jakobsen, M. Am. Soc. C. E., continued from September, 1926, *Proceedings*.

† Author's closure.

‡ Engr. in Chg. of Dams, Los Angeles County Flood Control Dist., Los Angeles, Calif.

§ Received by the Secretary, November 15, 1926.

‖ *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 241.

¶ *Loc. cit.*, p. 244.

** "Stabilité des Constructions," J. Résal, Librarie polytechnique, Paris, 1901, pp. 357 and 370.

†† *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 245.

‡‡ *Loc. cit.*, p. 242.

The second objection is much more serious. In Fig. 51 the points, A and B, lie just inside the secondary arch, while the points, A' and B', lie just outside. If the secondary arch carries all the load, the distance between the points, A and B, is decreased when the load is applied, but the distance between the points, A' and B', remains the same, since there is no stress outside the secondary arch. This is not possible and the error involved is likely to increase as the ratio of thickness of original arch to that of secondary arch increases. For short thick arches the maximum stress found for the secondary arch is, therefore, likely to be much in excess of the actual maximum stress.

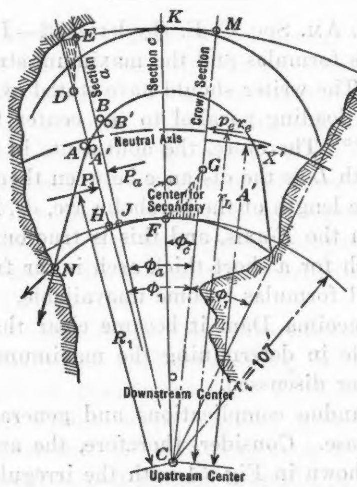

FIG. 51.—PACOIMA DAM AT ELEVATION 1700.

In addition, the direction of the maximum stress is not the same for the secondary arch as for the original arch and in order to judge intelligently of the sufficiency of the abutments, it is necessary to know something about the direction of that stress, which at Point N in Fig. 51 will be tangent either to the original or to the secondary arch. Since there is compression in the intrados near the abutment, it is not reasonable to assume, as is done for the secondary arch, that the concrete lying down stream from the secondary arch does not transmit stresses. In order to arrive at an understanding of what actually happens, the following calculations were made.

Neglecting the vertical stress due to the weight of the concrete and the water pressure on the up-stream face and, also, the effect of swelling, shrinkage, and temperature, as well as the deformation of the abutments, and assuming Poisson's ratio, $\frac{1}{m} = 8$, the resulting tangential stresses at the extrados and intrados at crown and at abutment can be determined. The constants for this arch are; $2\,\phi_1 = 41°$; $r_m = 149$ ft.; $t = 84$ ft.; $r_e = 191$ ft.; $r_i = 107$ ft.; $\frac{t}{r_m} = 0.564$; and, from Fig. 11,* $100\,\frac{c}{r_m} = 2.7$, or $c = 4.025$ ft., so that $r_n = r_m$

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 226.

$-c = 144.975$ ft. From Equation (18),* $A' = 141.904$ ft.; from Equation (38),† $\xi' = -2.17 \frac{p_e}{E}$; and, as a check, from Equation (38b),‡ $\xi'' = -2.2273 \frac{p_e}{E}$. The constant, B, is found by interpolation from Table 1§, $B = 0.392$, and Equation (58)‡ gives $X' = 162.7\ p_e$. From Fig. 12§, $k_e = 5.0$ and $k_i = 7.35$; p_e at Elevation 1 700 is due to 325 ft. of water on the assumption that the water stands level with the crest and that cantilever action is neglected, so that $p_e = 141.0$ lb. per sq. in. The stresses due to X' acting a distance, A', from the center of the arch may now be found from Equation (34)‖; they are, compression being negative, as given in Table 15.

TABLE 15.

Section.	Extrados.	Intrados.
Crown section	$+1.12\ p_e$	$+3.14\ p_e$
Abutments	$+2.08\ p_e$	$+1.42\ p_e$

The Lamé stresses from Equation (36)‖ are, for the extrados and intrados, respectively, $-1.92\ p_e$ and $-2.92\ p_e$. The resulting stresses are as given in Table 16.

TABLE 16.

Section.	Extrados.	Intrados.
Crown section	$-0.80\ p_e$	$+0.22\ p_e$
Abutments	$+0.16\ p_e$	$-1.50\ p_e$

By assumption the concrete cannot transmit tension and, consequently, the stress distribution found in Tables 15 and 16 cannot exist since it involves tension.

The tension in the extrados decreases from the abutment toward the crown, so that a point on the extrados close to the abutment can be found where the stress is zero; points closer to the crown section have compression. Likewise, there is a point on the intrados close to the crown section, where the stress is zero. Referring to Fig. 51, denote these sections as a and c; their location is determined from Equations (34) and (36):

$$\left.\begin{array}{ll} \text{For Section } a, & \sigma_{te}' + \sigma_{te}'' = 0 \\ \text{For Section } c, & \sigma_{ti}' + \sigma_{ti}'' = 0 \end{array}\right\} \dots\dots\dots\dots\dots (128)$$

The solution of Equations (128) gives the values of ϕ_a and ϕ_c (see Fig. 51).

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 215.

† *Loc. cit.*, p. 219.

‡ *Loc. cit.*, p. 224.

§ *Loc. cit.*, p. 226.

‖ *Loc. cit.*, p. 218.

The arch will yield in tension without resisting, approximately as indicated in Fig. 51 by the hatched areas. No tension exists in the arch between the two sections, a and c, and this part of the arch is therefore not affected.

Assume, now, that X' and A' remain unchanged and consider the part of the arch between Section a and the abutment. The shear forces acting radially in these two sections as well as the water pressure acting on the extrados, produce no moment about the center, C, of the arch. If P_a is the resultant of all tangential stresses acting on Section a and R_a is the distance from P_a to the center, C, and if P_1 and R_1 apply to the abutment section, it is,

$$P_a R_a = P_1 R_1 = M_c = \text{constant} \qquad (129)$$

Also, from Fig. 51, when P is the sum of all the tangential stresses on any section $\phi°$ from the crown section,

$$\left.\begin{array}{l} P = p_e r_e - X' \cos \phi \\ M_c = p_e r_e r_L - X' A' \end{array}\right\} \qquad (130)$$

in which, r_L is the distance from the center, C, to the point of application of the Lamé stresses. For any section the Lamé stresses must total $p_e r_e$ (see, also, Professor Cain's discussion*) and the lever arm, r_L, is obtained from Equation (36),

$$r_L = \frac{\int_{r_i}^{r_e} \sigma_t'' \, r \, d r}{p_e \, r_e} = \frac{r_e}{2}\left(1 + \frac{r_i^2}{r_m \, r_n}\right) \qquad (131)$$

On the assumption that X' and A' remain unchanged, the stress distribution for Section a is known and from Equations (129), (130), and (131), P_1 and R_1 may be determined. This gives the sum of the tangential stresses, or the thrust at the abutment and the point of application of this thrust, but it does not give the stress distribution, nor the thickness, $t' = DH$, and, therefore, the maximum stress that occurs in Point H is not determined.

It should be noted that the calculations are carried out for the condition that the up-stream and down-stream faces are concentric, which is true for the arc through the points, F, J, and H; the down-stream face is not concentric to the up-stream face and is shown as the arc through the points, F and N. Also, the calculations assume that the arch is symmetrical so that the radial line, CHD, is the abutment. If symmetry cannot be assumed, the calculations become considerably more involved.

The stress distribution near the abutment might be determined by making use of the fundamental Equations (71)† and (72)‡, by assuming the stress distribution as given by Equations (36) and (68).§ This would certainly be very complicated and not quite correct unless the abutment deformations were also taken account of (see Dr. Vogt's discussion, especially Fig. 45‖). The writer does not feel equal to the task of determining the stress distribution by calculation, and the only other avenue open is to select one or more

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 267.

† *Loc. cit.*, p. 236.

‡ *Loc. cit.*, p. 237.

§ *Loc. cit.*, p. 234.

‖ *Loc. cit.*, August, 1926, Papers and Discussions, p. 1254.

plausible assumptions and see how these affect the maximum stress that occurs in the point, H. Two assumptions will be investigated:

I.—That the tangential stress distribution at the abutment section, DH, is similar to that of Section a. The stress in the points, D and E, is zero.

II.—That the tangential stress distribution at the abutment section is linear.

Assumption I.—

Let,

$$\left.\begin{aligned} R_a &= r_i + k\,t \\ \text{and} \quad R_1 &= r_i + k\,t' \end{aligned}\right\} \qquad (132)$$

Since by assumption the stress distribution is similar for both sections, the coefficient, k, is a constant. M_c and P_a may be obtained from Equations (130) and R_a from Equation (129) and, then k follows from the first equation in Equations (132). Likewise, P_1 and R_1 are obtained from Equations (130) and (129) and, therefore, t' can be found from the last equation in Equations (132). Designate the stress in the points, J and H, of Fig. 51* by σ_J and σ_H, then,

$$\sigma_H = \sigma_J \frac{t}{t'} \frac{P_1}{P_a} \qquad (133)$$

The angle, ϕ_a, is obtained from the first equation in Equations (128),

$$\cos \phi_a = \frac{k_e \dfrac{A'}{t} + \sigma_{te}'' \dfrac{t}{A'}}{k_e \dfrac{r_m}{t} - 1} \qquad (134)$$

Applying Equation (134) to the arch under investigation gives $\cos \phi_a = 0.94744$, or $\phi_a = 20°\ 30'$; σ_J is obtained from Equations (34) and (36),

$$\sigma_J = -\ 1.21\ p_e$$

from Equation (131),

$$r_L = 146.12 \text{ ft.}$$

and from Equations (130),

$$M_c = 191 \times 146.12\ p_e - 162.7 \times 141.904\ p_e = 4\,821.14\ p_e$$

and,

$$P_a = 191\ p_e - 162.7 \times 0.94744\ p_e = 36.85\ p_e$$

From Equation (129),

$$R_a = \frac{4\,821.14}{36.85} = 130.86 \text{ ft.}$$

from Equations (132),

$$k = 0.28405$$

from Equations (130),

$$P_1 = 38.6\ p_e$$

from Equation (129),

$$R_1 = 124.9'$$

* Assuming that the arc, FJH, is the down-stream face.

and from Equation (131),

$$t' = 63.02'$$

The maximum stress at the abutment, which occurs at the point, H, is,

$$\sigma_H = -1.21\ p_e \frac{84}{63}\ \frac{38.60}{36.85} = -1.69\ p_e$$

The stress as computed by the cylinder formula is,

$$-\frac{191\ p_e}{84} = -\ 2.273\ p_e$$

or 35% greater than the stress, σ_H, which is the maximum stress on the assumption that X' and A' remain unchanged when the original arch yields in tension, as indicated in Fig. 51.

Assumption II.—Since now the stress distribution is linear,

$$\left.\begin{aligned} R_a &= r_i + \frac{t'}{3} \\ \text{and,}\qquad \sigma_H &= -\frac{2\ P_1}{t'} \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots\ldots(135)$$

R_1 was 124.9 ft., so that now, $t' = 53.7$ ft., as against 63.02 ft. found by Assumption I. P_1 was 38.60 p_e, so that now

$$\sigma_H = -\frac{2\ 38.6}{53.7} = -\ 1.44\ p_e$$

For linear stress distribution both t' and the maximum stress are smaller than under Assumption I.

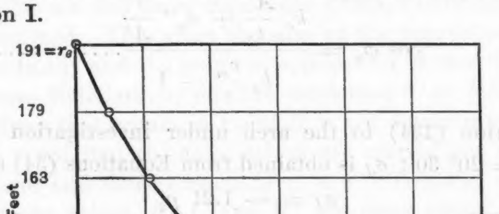

FIG. 52.—RESULTING TANGENTIAL STRESSES ON SECTION *a* OF FIG. 51.

In order to form an idea of the tangential stress distribution at Section *a*, this is shown in Fig. 52 and Table 17. The Lamé stresses are obtained from Equation (36) and the bending stresses from Equation (31),* in which,

$$M = X'\ (A' - r_m \cos \phi_a) = 119.58\ p_e$$

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 218.

The moment of inertia is $I = 49\,392$, so that $\frac{M}{I} = \frac{p_e}{413.05}$, and the tensile stress is,

$$\frac{X' \cos \phi_a}{t} = 1.835\, p_e$$

TABLE 17.—TANGENTIAL STRESS DISTRIBUTION ON SECTION *a* IN FIG. 51.
(Compression is negative.)

r, in feet.	Stresses in Terms of p_e.			
	Lamé.	Bending.	Tensile.	Total.
191	− 1.913	0.0846	+ 1.835	+ 0.0066
177	− 1.988	0.0631	"	− 0.0899
163	− 2.084	0.0388	"	− 0.2102
149	− 2.207	0.0094	"	− 0.3626
135	− 2.371	− 0.0260	"	− 0.5620
121	− 2.595	− 0.0696	"	− 0.8296
107	− 2.913	− 0.1248	"	− 1.203

The calculations that were made previously for Section *a* near the abutment were repeated for Section *c* near the crown section. The results were found to be, $\cos \phi_c = 0.99193$, or $\phi_c = 7° 17'$; this is much larger than the angle, $\phi_1 - \phi_a$, because at the abutments the moment changes rapidly while at the crown it changes slowly (see, Professor Cain's discussion, Fig. 32).* The stress at Point K for Assumption I is found to be, $-0.68\, p_e$ and $P_c = 29.613\, p_e$; $R_c = 162.80$ ft.; and $k = 0.33571$, when Equations (132) is replaced by,

$$\left.\begin{aligned} R_c &= r_e - k\,t \\ \text{and,} \quad R_o &= r_e - k\,t' \end{aligned}\right\} \dots\dots\dots\dots\dots\dots\dots\dots (132a)$$

At the crown-section, $P_0 = 28.30\, p_e$; $R_0 = 170.36$ ft.; $t' = 61.48$ ft.; and $6_M = -0.887\, p_e$; this is the maximum stress at the crown section.

In comparison the secondary arch as determined from Fig. 21, the dimensions of which have been previously given, has a maximum stress as determined from Equation (80)† of $5.35\, p_e = 738$ lb. per sq. in., since $p_e = 141$ lb. per sq. in. Under Assumption I the maximum stress was $1.69\, p_e$, so that the maximum stress of the secondary arch is 4.36 times greater. The writer stated specifically‡ that the stress found in the secondary arch was not the true stress, but a safe stress to use, especially as a first approximation.

The stress distribution thus found is predicated on the assumption that X' and A' remain unchanged when the hatched areas in Fig. 51 yield in tension without resisting. The writer believes that this assumption is approximately correct for the arch under consideration. He has made some calculations which seem to substantiate this belief, but they are of course only roughly approximate and can furnish only an indication; moreover, they involve addi-

* *Proceedings,* Am. Soc. C. E., February, 1926, Papers and Discussions, p. 262.
† *Loc. cit.*, p. 243.
‡ *Loc. cit.*, p. 241.

tional assumptions and since they are tedious and of no particular significance, they will not be given here.

In the stress calculations previously given it was assumed that the line, *H D,* coincides with the abutment and this is not true (see Fig. 51). The deformation and consequent yielding of the abutments were also neglected. It is evident that this subject needs considerably more study and the assumption of symmetry, which so materially simplifies the calculations, may have to be discarded. At any rate it needs to be demonstrated that the assumption of symmetry does not lead to large errors. An examination of Fig. 51 shows plainly that no line of symmetry exists and this is also the case at most other elevations for the Pacoima Dam, and, no doubt, is generally so. Also, it will be found that when a line of symmetry is drawn for each elevation, these lines do not lie in one vertical plane.

It should be noted that the line, *E D,* of zero stress in Fig. 51, is determined from the assumption that *C H D* is the actual abutment. Equations (129) and (130) can not be applied to find the stress distribution on a section to the left of Section *C H D,* because such procedure involves the assumption that the central angle is larger than that assumed in determining A' and X' and both A' and X' depend on the central angle. It would possibly have been more nearly correct to have assumed the central angle to be 10 to 15° larger, but the writer believes this would have given slightly smaller maximum stresses, since, at least, in general the maximum stresses decrease when the central angle is increased. In reality, the line, *E D,* of zero stress in Fig. 51 lies farther to the left and probably entirely inside the rock abutment.

General Description.—Pacoima and Santa Anita Dams.—The subject matter of the paper is largely confined to mathematical theory; some engineers might even go so far as to say it had nothing to do with actual dam design. As a matter of fact, the actual construction is intimately bound up with the design and unless the dam is so constructed, that the assumptions underlying the theory are actually fulfilled, the whole is unrelated and may be worthless.

The principles here developed have been used to advantage in both the Pacoima and Santa Anita Dams now being built for the Los Angeles County Flood Control District. To illustrate, therefore, the relation between theoretical investigations and practical construction a brief general idea of these two dams will be given, together with a description of some of the methods of construction.

In the Pacoima Dam* (Fig. 53) contraction joints are provided 50 ft. apart and are made in 5-ft. sections across the dam such that each 5-ft. section is offset tangentially 12 in. with reference to its two adjoining 5-ft. sections. In this manner at least one-half the section must be sheared across before a failure due to shear can occur. Each 5-ft. section is provided with a 2-in. split grout pipe rising vertically and these are connected horizontally every 50 ft., so that the contraction joint can be pressure-grouted later. An asphalt water-stop is provided near the up-stream face, and a V-shaped copper strip near the down-stream face seals the contraction joint. Pouring was begun

* This dam is the highest attempted to date.

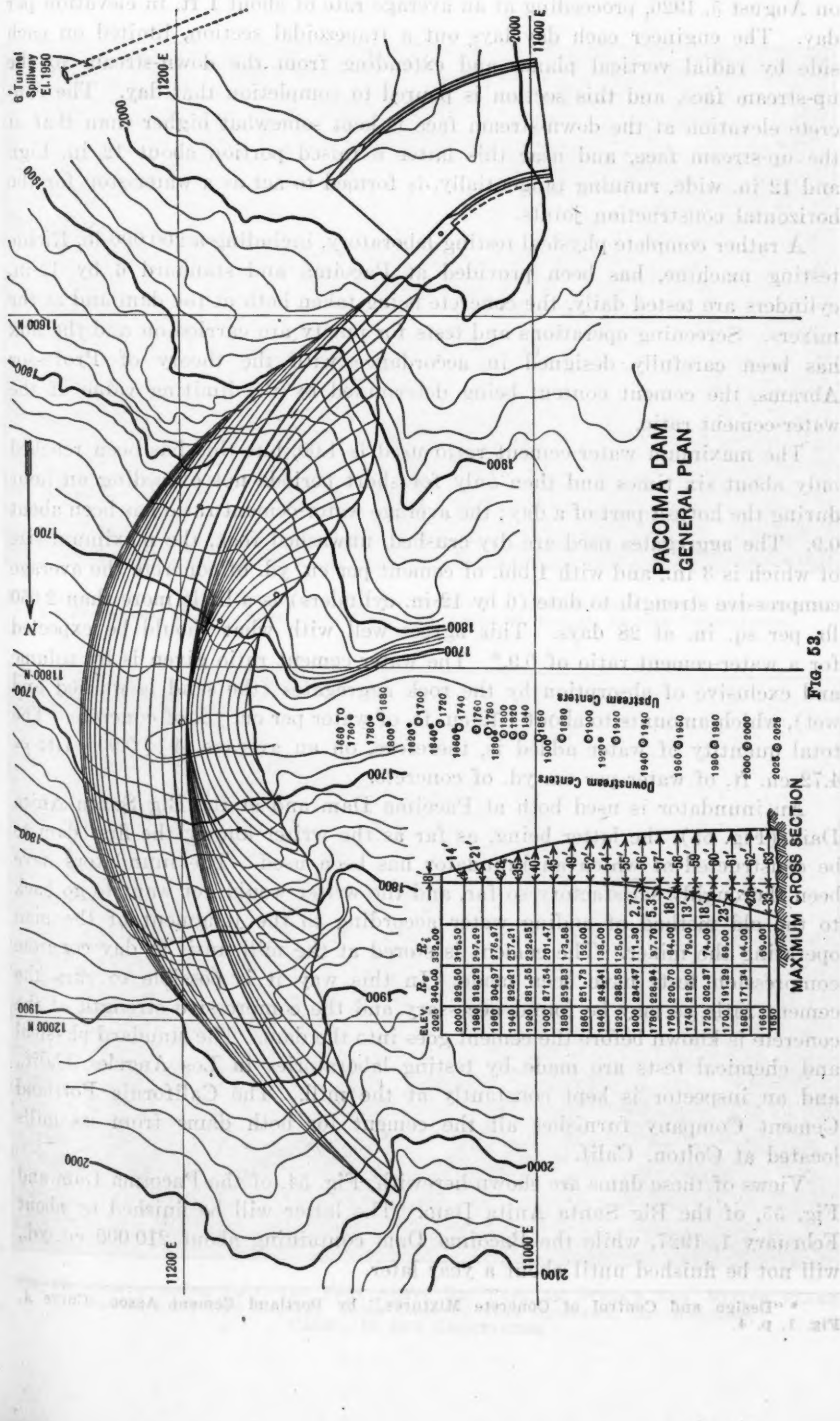
PACOIMA DAM
GENERAL PLAN
MAXIMUM CROSS SECTION
Upstream Centers
Downstream Centers
6' Tunnel Spillway El. 1950
N

FIG. 53

on August 5, 1926, proceeding at an average rate of about 1 ft. in elevation per day. The engineer each day lays out a trapezoidal section, limited on each side by radial vertical planes and extending from the down-stream to the up-stream face, and this section is poured to completion that day. The concrete elevation at the down-stream face is kept somewhat higher than that at the up-stream face, and near this latter a raised portion about 12 in. high and 12 in. wide, running tangentially, is formed to act as a water-stop for the horizontal construction joints.

A rather complete physical testing laboratory, including a 200 000-lb. Riehle testing machine, has been provided at Pacoima and standard 6 by 12-in. cylinders are tested daily, the concrete being taken both at the dam and at the mixers. Screening operations and tests for purity are carried on and the mix has been carefully designed in accordance with the theory of Professor Abrams, the cement content being determined by the limiting value of the water-cement ratio.

The maximum water-cement ratio used is 1.00, but this has been reached only about six times and then only for short periods not exceeding an hour during the hottest part of a day; the average water-cement ratio has been about 0.9. The aggregates used are dry-crushed, unwashed rock, the maximum size of which is 3 in., and with 1 bbl. of cement per cu. yd. of concrete the average compressive strength to date (6 by 12-in. cylinders) is a little more than 2 650 lb. per sq. in. at 28 days. This agrees well with what should be expected for a water-cement ratio of 0.9.* The water-cement ratio given is by volume and exclusive of absorption by the rock aggregates (the sand is washed and wet), which amounts to about 1.12 cu. ft. of water per cu. yd. of concrete. The total quantity of water added is, therefore, on an average, $4 \times 0.9 + 1.12 = 4.72$ cu. ft. of water per cu. yd. of concrete.

An inundator is used both at Pacoima Dam and at the Big Santa Anita Dam (Fig. 55), the latter being, as far as the writer knows, the first dam to be constructed at which an inundator has been used. The inundators have been thoroughly satisfactory so far, and the writer would not want to go back to the old method of adding water according to the judgment of the man operating the mixer. The cement is stored at the mill until 28-day concrete compression tests have been made. In this way it is possible to vary the cement ratio as may be found necessary and the compressive strength of the concrete is known before the cement goes into the dam. The standard physical and chemical tests are made by testing laboratories in Los Angeles, Calif., and an inspector is kept constantly at the mill. The California Portland Cement Company furnishes all the cement for both dams from its mills located at Colton, Calif.

Views of these dams are shown herewith, Fig. 54, of the Pacoima Dam and Fig. 55, of the Big Santa Anita Dam. The latter will be finished by about February 1, 1927, while the Pacoima Dam containing about 210 000 cu. yd., will not be finished until about a year later.

* "Design and Control of Concrete Mixtures," by Portland Cement Assoc., Curve A, Fig. 1, p. 4.

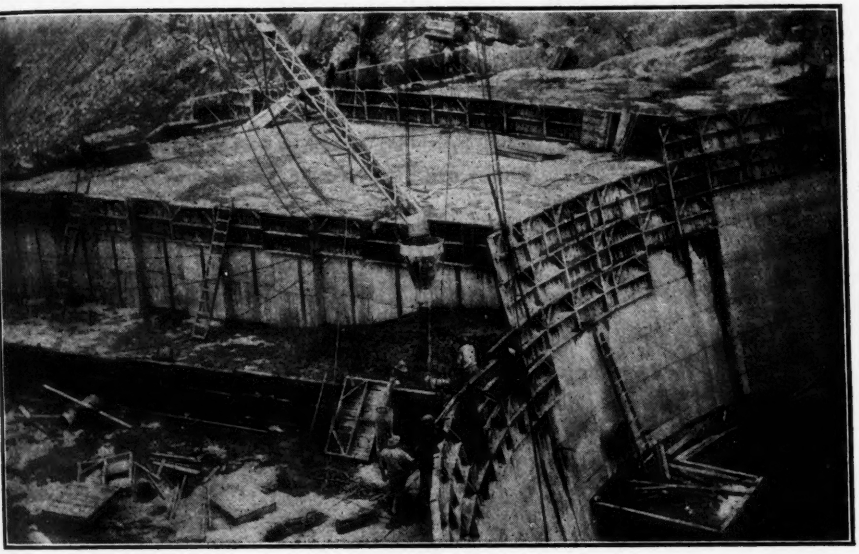

FIG. 54.—PACOIMA DAM, OCTOBER 6, 1926, SHOWING A CONTRACTION JOINT AND PART OF DOWN-STREAM FACE. BENT BROTHERS, LOS ANGELES, CALIF., IS THE CONTRACTOR.

FIG. 55.—SANTA ANITA DAM, 150 FEET ABOVE BED-ROCK IN STREAM BED, MIXING PLANT TO THE LEFT NOT SHOWN. ROSS CONSTRUCTION COMPANY, LOS ANGELES, CALIF., IS THE CONTRACTOR.

Influence of Swelling.—Referring to the original arch (Fig. 51), assume swelling of the up-stream face due to water-soaking and that the lateral deformation due to the weight of the concrete and the downward water pressure on the up-stream face is sufficient to compensate for shrinkage and temperature effects. Then (see Article XI*), let $u_s = 0.0001$ and $u_t = 0$, so that from Equation (48),† $\theta = 0.813 \times 10\,000$ and from Equation (49),† $y_c = 0.003595$; from Equations (50)† and (51)† are found,

$$\theta = -\frac{8.20367}{1\,000}\frac{X_c}{E}(A_c - 141.904)$$

and,

$$y_c = \frac{X_c}{E}-(0.00322551\ A_c + 0.2154425)$$

Inserting the values of θ and y_c just found and solving the two equations gives, $A_c = 143.776$ ft. and, if $E = 2\,000\,000 \times 144$ lb. per sq. ft., $X_c = -1\,575\,000$ lb. acting on an arch slice 1 ft. thick vertically. The stresses produced by X_c acting a distance, A_c, from the center are from Equation (34), compression being negative, as given in Table 18.

TABLE 18.

Section.	Stress at extrados, in pounds per square inch.	Stress at intrados, in pounds per square inch.
Crown section	−89.65	−189.8
Abutment	−154.5	− 74.0

Adding these to the stresses previously found for the original arch (see Tables 15 and 16), and since $p_e = 141$ lb. per sq. in., the resulting stresses obtained including the effect of swelling, are as given in Table 19.

TABLE 19.

Section.	Stress at extrados, in pounds per square inch.	Stress at intrados, in pounds per square inch.	Remarks.
Crown section	−112.9	+ 31.1	Original arch.
	− 89.7	−189.8	Due to swelling.
	−202.6	−158.7	Resulting stress.
Abutment	+ 22.6	−211.8	Original arch.
	−154.5	− 74.0	Due to swelling.
	−131.9	−285.8	Resulting stress.

There is compression throughout. The original arch shows a maximum tension at the intrados of the crown amounting to 31.1 lb. per sq. in. and this

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 222.

† *Loc. cit.*, p. 223.

is converted to a compression of 158.7 lb. per sq. in. It is quite evident that swelling is not even approximately negligible, but the writer does not claim that the assumptions here made cover the facts. Very little is known about the effect of swelling and shrinkage in a great body of concrete, but that does not warrant the assumption that it is negligible.* The writer had hoped to install some resistor type of cartridges for measuring deformations in both the Pacoima and the Big Santa Anita Dams, but was unable to obtain the necessary funds. He hopes other engineers may have better luck, since the questions involved are of considerable importance. Also, the writer felt the cartridges would afford an indication of how successful the pressure grouting was and, therefore, a guide as to how this grouting should be carried out in order to secure maximum effectiveness. As a direct result of the theoretical investigations it was possible to eliminate about $60 000 worth of steel from the Pacoima Dam.

Discussions.—Professor Cain's discussion constitutes, as is usual with him, a valuable contribution to a subject in which he has so largely pioneered, and the check afforded is, of course, gratifying. The writer prefers his own method of development, because it permits him to form a mental picture of the operation of the stresses and deformations, but Professor Cain is quite right when he insists on deriving the formulas by what he calls the "work method," also known as the principle of least work. This principle is so fundamental, that the entire theory of structures may be derived from it without further assumptions, except Hooke's law (proportionality between stress and strain), which is a generalization of experimental results rather than a theoretical assumption. The ordinary bending theory is based on Mariotte's assumption that normal sections remain plane. This is strictly true only when E is constant and when shear is not involved. By the method of least work the stress distribution due to a moment may be found by variational calculus and without making Mariotte's assumption.† When using the principle of least work, however, care must be taken in deriving the formula for work, since any assumption made for this purpose naturally involves the final result.

An interesting illustration of how an assumption is sometimes unwittingly introduced into reasoning is given by Föppl.‡ It was desired to compute the deflection of a piece of bent tubing. The result obtained by applying the standard bending theory gave deflections which were only about one-fifth the measured deflections. On examination it was found that the implied assumption that the section of the tubing was not deformed, was the cause of the discrepancy. A theoretical investigation, which made use of the principle of least work as applied by Ritz§ and took account of the fact that the section of the tube did not remain circular, led to a correct formula.

Professor Cain takes exception to the use of the Lamé formulas‖ and the following is intended to meet his criticism. In order to determine the stresses

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 270.

† *Loc. cit.*, Vol. LXXXVII (1924), p. 335.

‡ "Drang und Zwang," Bd. I, Second Edition, p. 70.

§ "Stresses in Multiple-Arch Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 325.

‖ *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 266.

in a cylinder subjected to external pressures, three different assumptions may be made:

1.—Neglect the radial and vertical stresses and deformations, that is, assume Poisson's ratio, $\frac{1}{m}$ = zero, and assume that the curvature of the arch is so slight that the arch may be treated as a straight beam.

This assumption was made by Professor Cain in his paper, "The Circular Arch Under Normal Loads".* It is not explicitly stated, but is indicated by his work formula, Equation (5), which neglects curvature, and by Equation (6), which neglects the rotation brought about by the normal stresses. Professor Cain at that time limited his investigation to a ratio of thickness of arch to mean radius, $\frac{t}{r} = 0.3$, and Assumption 1 may be considered a sufficiently close approximation for such arches.

It may be well to insist here that the reasoning in Professor Cain's paper, "The Circular Arch Under Normal Loads", is impeccable and it is not now a question of whether or not Professor Cain's reasoning was correct, but whether the assumptions he made are sufficiently close to the truth to yield reliable results, that is, results that will accord fairly well with actual experience. In this connection it should also be emphasized that whether or not the fundamental assumptions made in deducing a formula are acceptable for a particular case occurring in actual practice, this is something the engineer himself must judge of and for which the originator of the formula can in no wise be held responsible. It is the ability to judge in this matter which distinguishes an engineer as a man of scientific training and attainments from the mere artisan, who applies formulas mechanically and without understanding.

2.—Neglect the radial and vertical stresses and deformations, as under Assumption 1, but take account of curvature.

Assumption 2 leads to Professor Cain's revised formulas, as given in his discussion of the writer's paper (see, the work formula, Equation (98)).† In a footnote to this formula Professor Cain states that only the work of the radial stresses is omitted; this is not exact, since also the lateral deformations are neglected, that is, Poisson's ratio is assumed as zero. A comparison between this Equation (98) with Equation (5)‡ of Professor Cain's paper, "The Circular Arch Under Normal Loads", shows that the only change is from r, the mean radius, to r_n, the radius of the neutral axis. The writer's Fig. 11 shows that r_n and r_m are practically alike for small values of $\frac{t}{r_m}$.

3.—Radial as well as vertical stresses and deformations can not be neglected, nor can curvature.

Assumption 3 leads to the Lamé stresses, used by the writer and to Equation (68) for the stresses, in which, as outlined in Article XVI,§ the force, X,

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

† *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 258.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 236, and p. 268, where the influence of shear is added.

§ *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 236.

should be determined from Equations (68), (71) and (72). Since this led to complications the writer compromised and used the Lamé stresses combined with the stresses obtained from Equation (68), in which, X' was derived as shown in Article VI* by neglecting the radial stresses and deformations due to X. The error committed is believed to be small, because stresses found from Equation (34) check quite closely those obtained from Equation (68), when the value of X is—as assumed to be—the same in both cases.

Assumption 1 implies that the force, $p_e\, r_e$, which holds the water pressure in equilibrium, acts in the middle point of the section a distance, r_m, from the center, C, of the arch. This produces a uniform compression, $-\, p_e \frac{r_e}{t}$. The resulting stress is found by adding to it the stress due to the force, X, acting at the crown section. Assumption 2, which Professor Cain accepts, implies that the force, $p_e\, r_e$, acts at the neutral axis a distance, r_n, from the center, C. This is equivalent to a force, $p_e\, r_e$, acting in the middle of the section and producing a uniform compression, plus a moment, $M = p_e\, r_e\, (r_m - r_n)$, producing compression at the intrados. The stress due to this moment is given by the writer's Equation (31) and also by Professor Cain's Equation (93).† Combining this uniform stress with the bending stress leads to Professor Cain's Equation (97a),‡ in which, $P = p_e\, r_e$. To these stresses must be added those produced by X or X', which are determined in the same manner by both Professor Cain§ and the writer.‖

The only difference between Professor Cain's results and those of the writer is due to the difference between Assumptions 2 and 3; Professor Cain calculates the stresses due to water pressure by Equation (97a), while the writer used the Lamé stresses. That there is not a great deal of difference, except for very thick arches,¶ is shown by Table 20, which was calculated for the short thick arch of Fig. 51. The stresses are shown in Fig. 56. The maximum differences occur at the faces and amount to only 11 and 12.4 per cent.** When the maximum stress (compression) is computed (page 244), the difference between the two values is quite small and either value is acceptable.

Professor Cain states:††

"The only stresses then to be considered in this sliding stage [see Fig. 2]‡‡ are the uniform stresses, $\frac{R}{t}$, for which plane [radial] sections remain plane after strain, so that the sliding can be effected."

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 215.

† *Loc. cit.*, p. 254.

‡ *Loc. cit.*, p. 256.

§ *Loc. cit.*, p. 261.

‖ *Loc. cit.*, p. 217.

¶ *Loc. cit.*, p. 267; for the very thick arch, $\frac{t}{r_m} = 1.00$, the difference is considerable, but is due solely to the difference between Assumptions 2 and 3, as may be readily checked. Since tension is involved in both cases, the question arises as to how great the difference would be between the maximum stresses in the two cases, when these are determined as shown (see page 244). The writer has not checked this, but believes the difference would not be material.

** It should be noted that these stresses are not the final stresses; the stresses produced by X must be added, see, Table 17.

†† *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 267.

‡‡ *Loc. cit.*, p. 211.

TABLE 20.

r, in feet.	Stresses, in Terms of p_e.	
	Lamé, Equation (36).	Cain, Equation (97a).
191	− 1.913	− 1.725
177	− 1.900	− 1.863
163	− 2.084	− 2.022
149	− 2.207	− 2.212
135	− 2.371	− 2.440
121	− 2.595	− 2.725
107	− 2.913	− 3.160

The stresses required to cause only translation of a section is not a uniform stress, but is given by Professor Cain's Equation (97a), using Assumption 2, as he shows himself.* This was also shown by the writer in Articles V† and VI, in which the radial stresses and deformations were neglected. When, however, radial stresses and deformations are to be considered, the Lamé stresses are needed to conserve radial sections plane, as may be shown by applying

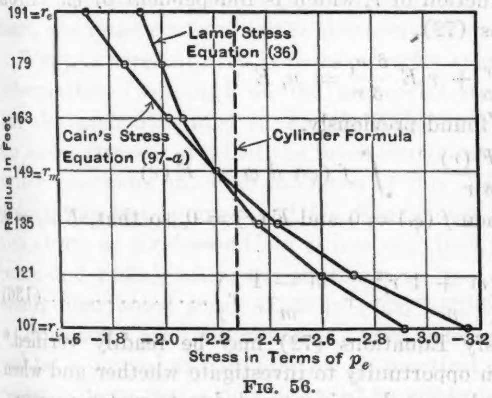

FIG. 56.

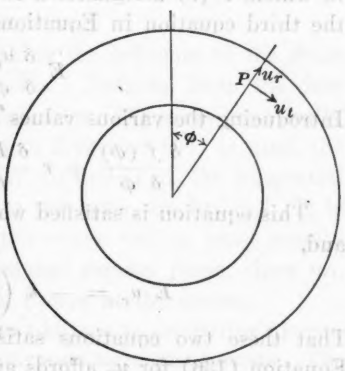

FIG. 57.

Equation (72) to the Lamé stresses. Referring to Fig. 57, u_r and u_t are the radial and tangential displacements of the point, P, positive in the directions indicated. Likewise, σ_r and σ_t are the radial and tangential stresses. The Lamé stresses are given by Equations (36) and (37),‡ and the shear is zero. If the Lamé stresses are introduced into the equilibrium, Equations (71), it will be found that these stresses satisfy Equations (71) and that Professor Cain's Equation (97a) does not, as already pointed out by the writer.§

From the Lamé stresses and the first equation of Equations (72),

$$E \frac{\delta u_r}{\delta r} = -\left(1 - \frac{r_i^2}{r^2}\right) k + \frac{1}{m}\left(1 + \frac{r_i^2}{r^2}\right) k$$

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 255.

† *Loc. cit.*, p. 214.

‡ *Loc. cit.*, p. 219.

§ *Loc. cit.*, p. 237.

in which,

$$k = \frac{r_e^2\, p_e}{r_e^2 - r_i^2}$$

Integrating toward r, gives,

$$E\, u_r = -\,k \left(\frac{m+1}{m}\frac{r_i^2}{r} + \frac{m-1}{m}\, r\right) f + (\phi)$$

in which, $f(\phi)$ designates a function of ϕ, which may be constant or zero, but does not contain r, that is, is independent of r. From the second equation in Equations (72) and the Lamé stresses,

$$E\left(\frac{\delta\, u_t}{\delta\, \phi} + u_r\right) = -\,k\left(\frac{m+1}{m}\frac{r_i^2}{r} + \frac{m-1}{m}\, r\right)$$

Substituting the value of $E\, u_r$ found previously,

$$E\,\frac{\delta\, u_t}{\delta\, \phi} = f(\phi)$$

or,

$$E\, u_t = \int f(\phi)\, d\,\phi + F(r)$$

in which, $F(r)$ designates a function of r, which is independent of ϕ. From the third equation in Equations (72),

$$E\,\frac{\delta\, u_r}{\delta\, \phi} + r\, E\,\frac{\delta\, u_t}{\delta\, r} = u_t\, E$$

Introducing the various values found previously,

$$\frac{\delta\, f(\phi)}{\delta\, \phi} + r\,\frac{\delta\, F(r)}{\delta\, r} = \int f(\phi)\, d\,\phi + F(r)$$

This equation is satisfied when $f(\phi) = 0$ and $F(r) = 0$, so that, $E\, u_t = 0$, and,

$$E\, u_r = -\,k\left(\frac{m+1}{m}\frac{r_i^2}{r} + \frac{m-1}{m}\, r\right) \ldots\ldots\ldots\ldots\ldots(136)$$

That these two equations satisfy Equations (72) may be readily verified.* Equation (136) for u_r affords an opportunity to investigate whether and when the thickness, t, of the arch is decreased or increased due to water pressure. Inserting $r = r_e$ and $r = r_i$, and subtracting, gives,

$$E\,(u_{re} - u_{ri}) = -\,k\left(\frac{m+1}{m}\frac{r_i^2}{r_e} + \frac{m-1}{m}\, r_e - 2\, r_i\right)$$

$$= k\left(\frac{2\,t}{m} - \frac{m+1}{m}\frac{t^2}{r_e}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots(137)$$

When this is positive the thickness, t, of the arch is increased. For $m = 8$, for example, the following holds: If $\frac{t}{r_e} = \frac{2}{9}$, t remains constant; if $\frac{t}{r_e} > \frac{2}{9}$, t decreases (thick arches);† and, if $\frac{t}{r_e} < \frac{2}{9}$, t increases (thin arches).

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* The assumption, $u_t = 0$, is made in developing the Lamé formulas, see, for example, Bach-Baumann, "Elasticitat und Festigkeit", Ninth Edition (1924), p. 571. The calculations just given show that this assumption is correct.

† *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 270.

For the arch shown in Fig. 51, it is,

$$u_{re} - u_{ri} = - 30.1 \frac{p_e}{E}$$

that is, t is decreased. If $m = \infty$, and the radial stress decreases linearly from p_e to zero, the contraction would amount to $42.0 \frac{p_e}{E}$. As a comparison, the displacement, $DF = e'$, in Fig. 2, is $r_n \cdot \varepsilon'$, and ε' was $- 2.17 \frac{p_e}{E}$* so that $e' = - 415.0 \frac{p_e}{E}$.

Professor Cain† discusses the influence of shear on tangential stresses; Equation (68) takes this influence into account, on the assumption that the arch is free to deform at the abutment as this stress distribution requires. In order to find the deformations, the writer made an attempt to solve Equations (72) when the stresses were those given by Equation (68). This would give the warping of the originally plane radial sections, but would not be a solution after all, because the arch is not free to warp its radial section at the abutment, but must conform to the abutment deformations.‡

What is here involved is more correctly speaking the influence of the shear deformation (warping) on the tangential stresses. Judging from the fact that the arch computed in Article XII§ by Equation (34) gives practically the same stresses as when the more nearly correct Equation (68) is used, the writer concludes that the influence of this shear distortion on the tangential stresses can not be great and Dr. Vogt comes to the same conclusion.‖ It is also clear, as Professor Cain points out, that the crown section must remain plane and radial, while the abutment section cannot remain plane, since unevenly distributed shear exists in every section except at the crown.

Professor Cain's discussion is a very welcome addition to the writer's paper, and he is pleased to note that for all practical purposes agreement exists. The writer believes that a dam designed by either formula will result in a safe and economical structure. He would like to add that, in his opinion, neither of the two Assumptions 1 and 2 are exact since Poisson's ratio is not zero and, on the other hand, the arch is not free to deform laterally at the abutments, as assumed in the Lamé formulas.

In order to find the true stresses in the arch in Fig. 51 or in the part lying between the abutment and Section a, the fundamental equations of Article XVI, as already stated, would have to be resorted to. It is quite likely that this would be very complicated and that Professor Cain's Equation (98) would lend itself much more readily and with quite sufficient accuracy to such a stress determination. The writer would like to suggest that Professor Cain make such an attempt since this would be of great value to the engineer in

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 270.

† *Loc. cit.*, p. 245.

‡ *Loc. cit.*, August, 1926, Papers and Discussions, p. 1246, especially Fig. 45, p. 1254; also, *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 237.

§ *Loc. cit.*, February, 1926, Papers and Discussions, p. 224.

‖ *Loc. cit.*, August, 1926, Papers and Discussions, p. 1253.

determining the actual maximum compression on the assumption that tension cannot be transmitted.

Professor Harris* brings up nothing new. That cantilever action ought to be considered is generally agreed, but how this can be done is the question, and Professor Harris contributes nothing to its solution. The writer is reminded of a remark by Lord Kelvin to the effect that "a phenomenon is well known, when it is known in figures", and there are no figures in Professor Harris' discussion. The writer tried some years ago to take account of the cantilever by using the method of least work as applied by Ritz,† but he came to the conclusion that the calculations, even if possible, were so extremely complicated as to be of little practical use, and that in order to render them fairly simple so many approximations would have to be made in the assumptions, that he had no confidence in the results. At that time he also read over the discussion of plates given by Föppl‡ which is quite involved despite the fact that he does not consider temperature influences, swelling, and shrinkage, but only plates in which $\frac{t}{r_m}$ may be assumed to be small. Add to that the fact that plates are generally symmetrical while dams are not and the resulting mathematical complications are obvious.

Professor Harris seems to consider only the cylinder formula and he concludes§ that no matter how close together the canyon walls are, a dam with a straight face carries all the load on the cantilever, while the truth is that it carries but very little load in that manner, since the load is carried by the horizontal secondary arches.‖ The cylinder formula is approximately correct for fairly thin long arches, but it is not a general formula and when used as such in reasoning it leads to absurdities. Referring to Fig. 2, if a uniform stress, σ, is applied to the semi-arch the displacement, $e = DF$, at the crown is,

$$e = -\int \frac{\sigma}{E} r \cos \phi \, d \phi = -\frac{\sigma}{E} r \sin \phi_1$$

and this is not zero except when $E = 0$. This is not an assumption that can be safely made, but one that must be shown to lead to a fair approximation in each specific case. It is also evident that if a sufficient number of simple assumptions be made, it is quite easy to determine the distribution of load between the cantilever and the arches, but this would afford merely an exercise in elementary mathematics and no competent engineer could be induced to trust to the results of such formulas.

Mr. Jorgensen¶ considers the formulas presented by Professor Cain and by the writer as somewhat elaborate for every-day use. These formulas are

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1239.

† "Stresses in Multiple-Arch Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 325.

‡ "Drang und Zwang," Bd. I, Second Edition, pp. 123 to 244.

§ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1240.

‖ "Formes et dimensions des grands barrages en maçonnerie," Résal, *Annales des Ponts et Chaussées*, p. 26, and, on p. 29, where he states that when the length of a straight dam equals 2.5 times the height, the arch action is insignificant, but if the length does not exceed one-third the height, it is the arch that carries all the load.

¶ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1245.

for use with thick arches, which are expensive. In the case of the Pacoima Dam, for example, the saving of the constant-angle arch dam over a gravity dam is in excess of $1 000 000, and for a small fraction of this amount some very extensive theoretical investigations could be and should be made. Mr. Jorgensen has possibly confused the work of deriving these formulas with that of applying them and while the writer knows nothing about bridge design, he doubts whether a careful theoretical investigation of the Pacoima Dam would require more time or skill than is required for the proper design of a bridge costing approximately the same, that is $2 000 000. The writer doubts very much whether it will ever be possible to obtain empirical formulas for dams of a sufficient range to be of much value. Tests on models, however, might be made valuable as a help to theoretical considerations.

Mr. Jorgensen refers to some tests on models, made by B. E. Torpen, M. Am. Soc. C. E., in connection with the construction of the Lake Cushman Dam. The tests showed, as Mr. Jorgensen states, that the water pressure required to break one of these arches, or perhaps several, would give rise to a tangential stress (computed by the cylinder formula), about 100% higher than the compressive stress found for the concrete when tested in ordinary 6 by 12-in. compression cylinders. Mr. Jorgensen then states: "* * * and it was demonstrated * * * that at or near the point of failure the stresses were distributed evenly over the section." As a matter of fact, this was not demonstrated, since no determination was made of the stress distribution at or near breaking; but it is Mr. Jorgensen's conclusion, and that is another matter. Mr. Jorgensen evidently reasons that since the stresses found in arches by Professor Cain's or the writer's formulas are higher than the cylinder stresses (and that is especially true of the stresses in the secondary arches given by the writer), and since the water pressure required to break these arches corresponds to a cylinder stress of 100% in excess of the strength of the concrete in 6 by 12-in. cylinders, the cylinder formula applies, at least more nearly than the others. However, 100% in excess is not a very satisfactory check and certainly such a divergence should not be taken as proof that the stresses were uniformly distributed.

The writer stated specifically in his paper, that the secondary arch stresses were not the true stresses, but were higher than the true stresses and, therefore, safe to use. He has already shown that the maximum stress in a thick arch is very likely less than the cylinder stress. The model arch did not break at the abutment, probably on account of lateral restraint, but the fact that the cylinder formula gives results that are 100% in excess of the compressive strength of the concrete, coupled with the fact that the arch did not break at the abutment, leads the writer to conclude that the cylinder formula is certainly incorrect.

The writer thoroughly agrees with Mr. Jorgensen that it is not safe to assume that any load is taken by the cantilever, for the reason that at present no method of obtaining the correct distribution is known.

Dr. Vogt's discussion* is a valuable contribution to the subject in that he makes it possible to take the yielding of the abutments into account with

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1246.

but little more work than is required when it is neglected. The writer had prepared a discussion of the influence of the yielding of the abutments based on Dr. Vogt's thesis*, but Dr. Vogt's discussion covers the subject much more ably. His criticism† of the writer's Equation (48) is just; the radial deformation was neglected. As already discussed in replying to Professor Cain's criticism,‡ it is not correct to neglect radial deformation, and on the other hand near the abutments the radial deformation is no doubt restrained to some extent, even if it be argued that the rock near the abutment can not vary much in temperature from the concrete near the abutment. The writer prefers Dr. Vogt's Equation (117)§ to his own Equation (48), but the difference in the results obtained by these formulas is probably less than the probable error of the temperatures, etc., involved.

Dr. Vogt's Equations (119*b*)§ and (121*b*)‖ take into account temperature effects, swelling, shrinkage, the lateral deformation due to the vertical stress, σ_x, and the deformation of the abutments. All things considered, these equations are remarkably simple. They may, however, be still further simplified by assuming the angle, $\psi_1 = 0$ (see Fig. 44),¶ which assumption he believes is usually approximately correct and when not correct the influence of this angle, ψ_1, is very difficult to determine, as Dr. Vogt states**; moreover this angle is not likely to be the same for both abutments (see, for example, Fig. 51). The writer also prefers to write the equations so that they are independent of the assumption that the modulus of elasticity for rock and concrete is the same, although this is probably nearly correct after the concrete is a year or two old. Johnson,†† for example, gives for Brandford (Conn.), granite, 8 333 300; for Milford (Mass.), granite, 6 663 000; and for Troy (N. H.), granite, 4 545 400; these moduli are for working loads for granite under compression and are taken from tests made at the Watertown Arsenal. Stanton Walker, Assoc. M. Am. Soc. C. E., has shown‡‡ that the rapid increase of the modulus with age, and, for example, for a concrete with aggregates up to 2 in., the modulus at 7 days is about 2 500 000 lb. per sq. in., while at 1 year, it is 7 000 000 lb. The writer also prefers to omit Dr. Vogt's constant, *q*, as defined in Equation (116)§§.

Let E_f equal the modulus of elasticity of the rock; and E that of concrete. Also, referring to Equation (112),‖‖ let, $\xi' = \xi \dfrac{E}{E_f}$; $\eta' = \eta \dfrac{E}{E_f}$; and $\mu' =$

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* "Ueber die Berechnung der Fundament Deformation," *Det Norske Videnskapsakademi*, Oslo, 1925, No. 2.

† *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1250.

‡ See p. 261.

§ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1250.

‖ *Loc. cit.*, p. 1251.

¶ *Loc. cit.*, p. 1249.

** *Loc. cit.*, p. 1252.

†† "The Materials of Construction," Fourth Edition (1912), p. 645.

‡‡ "Modulus of Elasticity of Concrete," *Bulletin 5*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., January, 1920, Edition, p. 41.

§§ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1250.

‖‖ *Loc. cit.*, p. 1248.

$\mu \frac{E}{E_f}$. In case $E = E_f$, $\xi' = \xi$, etc. Then, with the angle, $\psi_1 = 0$ (see Fig. 44), Dr. Vogt's Equations (119*b*) and (121*b*) can be written more simply, a follows:

$$X''' \left\{ r_n \sin \phi_1 + \mu' c \frac{r_m}{t} \cos \phi_1 - \eta' c \sin \phi_1 \right\} - X'' A'' \left(\phi_1 + \mu' \frac{c}{t} \right)$$

$$= p_e r_e \mu' \frac{c^2}{t} + \Delta u E c r_m \phi_1 \ldots\ldots\ldots\ldots\ldots\ldots (138)$$

and,

$$X'' \left\{ -0.5 r_n \left(2 \sin \phi_1 - \phi_1 - \frac{1}{2} \sin 2 \phi_1 \right) + c \left(c_1 \phi_1 - \frac{c_2}{2} \sin 2 \phi_1 \right) \right.$$

$$+ 12 \xi' \frac{c^2}{t} - \eta' c \sin \phi_1 \left(2 \frac{r_m}{r_n} \cos \phi_1 - 1 \right) + \mu' c \frac{r_m}{t} \left(\frac{r_m}{r_n} \cos \phi_1 - 1 \right) \cos \phi_1 \Big\}$$

$$+ X'' A'' \left\{ (\phi_1 - \sin \phi_1) + \eta' \frac{c}{r_n} \sin \phi_1 - \mu' \frac{c}{t} \left(\frac{r_m}{r_n} \cos \phi_1 - 1 \right) \right\}$$

$$= p_e r_e c \left\{ \sigma \sin \phi_1 + 12 \xi' \frac{c}{t} \cos \phi_1 - \eta' \frac{c}{r_n} \sin \phi_1 + \mu' \frac{c}{t} \left(\frac{r_m}{r_n} \cos \phi_1 - 1 \right) \right.$$

$$- 12 u_n c^2 \frac{r_n}{t} E \sin \phi_1 - \Delta u E c r_m (\phi_1 - \sin \phi_1) \ldots\ldots\ldots (139)$$

Equations (138) and (139) have been carefully checked and the example given by Dr. Vogt has been calculated to make sure that no error was committed in re-writing them. There is undoubtedly a considerable uncertainty in determining the coefficients, ξ, η, and μ in Equations (112). The values used by Dr. Vogt* are nearly the mean values from Equations (112), and these may no doubt be used quite generally.

In using Equations (138) and (139) some care must be taken since u_n in accordance with Dr. Vogt's Equation (120)† for e' includes the temperature effect, swelling, and shrinkage, besides the deformation due to the vertical stress, σ_x, which latter only is included in the writer's Equation (38) to which Dr. Vogt refers†. Equations (117) and (118)‡ should be used, but as stated, u_e and u_i contain also the deformation produced by σ_x, which is (see Equation (38)) $-\frac{\sigma_x}{m E}$. Since the stress, σ_x, does not produce rotation of radial sections, it is to be added to both u_e and u_i and, therefore does not affect θ from Equation (117). The two constants, c_1 and c_2, in Equation (139) are given by Dr. Vogt in the equations immediately following Equation (116) and the values used in the paper are, $c_1 = 1.94$ and $c_2 = 0.94$; and σ is obtained from Equation (120).

Dr. Vogt assumes the resultant, $R = p_e r_e$, of the water pressure as acting in the neutral axis (see Fig. 44 and Equations (114)§ for the moment,

* *Proceedings*, Am. Soc. C. E., August. 1926. Papers and Discussions, p. 1252; the mean values from Equations (112) are $\xi = 1.77$; $\eta = 0.68$; and $\mu = 5.34$.

† *Loc. cit.*, p. 1251.

‡ *Loc. cit.*, p. 1250.

§ *Loc. cit.*, p. 1249.

M_1, referred to the center of gravity axis), so that R produces a moment equal to $Rc = R\,(r_m - r_n)$. This is, as Dr. Vogt states, only approximate, and the writer thinks it sufficiently correct even if R actually acts at a point farther from the center, C, of the arch (see Fig. 44), namely, at a distance, r_L, as given by Equation (131). This could be taken care of by substituting for $p_e\, r_e\, c$, in Equations (138) and (139), $p_e\, r_e\, (r_m - r_L)$. However, this is not correct either, since this moment would not give the Lamé distribution of stresses and, therefore, the correction is hardly worth while. Dr. Vogt, as well as the writer, determined the stresses produced by $p_e\, r_e = R$ in the arch, from Equation (36).

When in Equations (138) and (139), the constants, ξ', η', and μ', are put equal to zero and the proper value for u_n is introduced (that is, $u_n = 0$, if no temperature effects, swelling, and shrinkage and no lateral deformation due to σ_x are to be considered), the writer's Equations (18) and (29a)* result, in which, X' is given by Equation (30)* and $\sigma_x = 0$ in Equation (38). If the influence of σ_x is to be included in Equations (138) and (139), then $u_n = -\dfrac{\sigma_x}{m\,E}$; here, u_n is positive since σ_x, being compression, is negative.

Equation (127), for the deflection of the arch when the yielding of the abutment is considered, is simple to use in connection with Fig. 46† and shows that the deflection in the example considered by Dr. Vogt is twice as great when the abutment yielding is considered as when it is assumed to be negligible, which it is not. Equation (127) is obtained by considering the arch to have a somewhat larger central angle than it actually has. This can not be used in connection with the stresses, since it would lead to a decrease in all stresses, while as a matter of fact the yielding of the abutment increases the tension at the intrados of the crown.‡ The writer hopes that Dr. Vogt will publish his proposed paper on the distribution of load between the arches and the cantilevers, when the yielding of the abutments of the arches and the foundations of the cantilevers are taken into account, as this would be a great step toward a rational solution of an important problem in dam design and, one which would likely lead to considerable savings in material. Dr. Vogt's excellent work has already prepared the way for this.

Dr. Vogt does well in emphasizing, as he does repeatedly in his discussion, that the results obtained from his calculations can only be roughly approximate; the same is true of most stress determinations in which anything but simple stresses are involved. This is due to the fact that the various assumptions which must be made in order to treat the subject mathematically and without too great complications, are only approximate. In order not to be misunderstood, it should also be emphasized and insisted on that this is not an admission that the engineer who reasons by "hunches" and from "common sense", and without any specific knowledge of the subject involved, is as likely to arrive at the correct or approximately correct result, as he who starts from fundamental assumptions and develops his subject mathematically, which is

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 217.

† *Loc. cit.*, August, 1926, Papers and Discussions, p. 1257.

‡ *Loc. cit.*, p. 1252; see, also, Table 12, p. 1253.

the only way it can be developed. Common sense is quite requisite, but it does not supersede specific knowledge* and, moreover, the engineer with some training in mathematics has as valid a claim on common sense as any one.

Mr. Paaswell's discussion† brings up some interesting points. He is right in objecting‡ to the writer's statement "that $\sigma_y = 0$, or that the Poisson ratio is zero." These are two different assumptions, as Mr. Paaswell points out, and what the writer should have said and intended to say, was that these two assumptions are often used together. The formula for determining the temperature variations given by Mr. Paaswell, and quoted from his work on retaining walls,§ checks quite well with the results of tests, and ought to prove of value to engineers in the design of dams. Mr. Paaswell considers a thin section of an arch barrel treating it as a surface. This development the writer cannot follow as he is not acquainted with Love's treatise. From Fig. 48‖ the writer gathers that since the moment in the arch at the crest is zero the crown deflection is zero, and this is not in agreement with what has been generally found for arch dams. Some tests made on the deformation of a cylinder,¶ however, show no deformation at the crest, and, of course, Mr. Paaswell's calculations are not claimed by him to be directly applicable to an arch dam, as usually built. If an equation could be developed for an arch with variable central angle and variable thickness, this might afford a means of analyzing arch action and cantilever action combined. This might be of value even if it should become necessary to resort to some simplifications of the assumptions, such, for example, as that the neutral and gravity surfaces (axes) coincide, as was assumed by Mr. Paaswell.** The writer sincerely hopes that Mr. Paaswell will return to this subject and make an attempt to apply his method of attack.

Mr. Perkins has been charged for a number of years with the investigation of designs of dams submitted to the State Engineer of California and in this capacity he has accumulated a considerable knowledge of the subject. The writer agrees with Mr. Perkins' discussion†† except perhaps that he places less confidence in the calculations of the division of load between cantilever and arches. Mr. Perkins' remarks concerning the secondary arches are true and have already been dealt with in a fashion, but there still remains to be proven that X' and A' remain approximately unchanged.

Mr. Perkins states‡‡ that the "uncertainty in the value of E does not have so much bearing because of the fact that E enters in the same way in both

* See Kant's scathing remarks in his "Prolegomena," edited in English by Dr. Paul Carus, The Open Court Publishing Co. (1909), Introduction, p. 6: "To appeal to common sense, when insight and science fail, and no sooner—this is one of the subtle discoveries of modern times, by means of which the most superficial ranter can safely enter the lists with the most thorough thinker, and hold his own. But as long as a particle of insight remains, no one would think of having recourse to this subterfuge."

† *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1259.

‡ *Loc. cit.*, p. 1260.

§ "Retaining Walls: Their Design and Construction."

‖ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1262.

¶ "Experimental Deformation of a Cylindrical Arched Dam," by B. A. Smith, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1596.

** *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1261.

†† *Loc. cit.*, September, 1926, Papers and Discussions, p. 1485.

‡‡ *Loc. cit.*, p. 1486.

cantilever and arch action, and, therefore, in a sense, is cancelled." This is not quite correct in large dams which require considerable time to construct, due to the fact that E increases with age, so that although E may be fairly constant for any horizontal arch, it varies from elevation to elevation, and is not constant for the cantilever, but may vary quite considerably, just as does the strength.* Also, the wetting of the up-stream face may be of considerable importance,† as well as the yielding of the abutments.‡ A study such as Mr. Perkins' Fig. 49, made in order to determine the probable division of load between the arches and the cantilever, is always of interest, but the engineer should bear in mind the fact that important factors have been left out of consideration and that, therefore, the results should not be taken too seriously.

Mr. Noetzli in his discussion§ remarks "that the author has introduced a number of assumptions * * *", while the truth is that the writer reduced the number of assumptions and it is this circumstance which leads to the greater mathematical complexity. In deriving the stresses by the Lamé formulas and Equation (68) there is assumed only that E and m are constants. In deriving Equation (34) it was further assumed that radial sections remain plane and that Poisson's ratio is zero. Finally, in deriving the cylinder formula, it is assumed that the stress distribution is uniform over the cross-section. The assumptions which Mr. Noetzli lists, he will find in the writer's paper as well as in his discussion of Mr. Noetzli's paper, "Gravity and Arch Action in Curved Dams".‖ Regarding Mr. Noetzli's quotation from the writer's discussion of the paper on "Gravity and Arch Action in Curved Dams", the writer devoted about eighteen pages to this and is quite content to rest his case there. He wishes to add, however, that if he were to design an arched dam and had the choice between the cylinder formula and Mr. Noetzli's theory, he would choose the cylinder formula for the reasons given in the aforementioned discussion. He might add that, according to Mr. Noetzli's theory, the Kerckhoff Dam, designed by the writer¶ should long ago have failed as it should have high tension in the heel, but this dam is still functioning and earning dividends.

Regarding Mr. Noetzli's statement** as to what Professor Cain thinks about deflections of arched dams, the writer would refer to Professor Cain's own statement.†† Rather than reply in detail to several of Mr. Noetzli's other statements, the writer would quote Henri Bergson, as given by Will Durant:‡‡

"I believe that the time given to refutation in philosophy is usually time lost. Of the many attacks directed by the many thinkers against each other,

* *Bulletin 5*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., January, 1921, Fig. 22, p. 41, E varies logarithmically with the age, for example, from 2 500 000 at 7 days to 7 000 000 at 1 year.

† *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 248.

‡ *Loc. cit.*, August, 1926, Papers and Discussions, p. 1256.

§ *Loc. cit.*, September, 1926, Papers and Discussions, p. 1489.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 92.

¶ *Loc. cit.*, p. 107.

** *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1489.

†† *Loc. cit.*, February, 1926, Papers and Discussions, p. 277.

‡‡ "The Story of Philosophy," 1926, p. 503.

what now remains? Nothing, or assuredly very little. That which counts and endures is the modicum of positive truth which each contributes. The true statement is of itself able to displace the erroneous idea, and becomes, without having taken the trouble of refuting any one, the best of refutations."

The writer believes Mr. Noetzli is somewhat too optimistic about what the tests on the Stevenson Creek Dam will bring forth. It is one thing to determine a few constants, as is done, for example, when the over-all efficiency of a power plant is determined, and it is quite another matter to derive a natural law, especially when this is to be done from a limited number of tests on a single structure. The writer speaks from experience, for in 1914 he made some tests on corona losses on a high-tension transmission line in the Peruvian Andes with a view to determining the law for these losses* and he, therefore, believes that he has a fair appreciation of the difficulties to be overcome. To that may be added that it is not directly obvious how tests on a relatively thin dam can furnish any information as to the stress distribution in thick dams. The upper part of the test dam is so thin and long that there seems to be danger of failure by buckling rather than failure in arch action. The writer has investigated the test dam, using the formula for buckling,† in which

$$p_c = \frac{E}{4}\left(\frac{t}{r}\right)^3$$

p_c is the critical water pressure, that is, the pressure at which buckling may be expected. He called this to the attention of one member of the Los Angeles Committee in charge of the tests several months before concrete pouring was started and he was told that the matter had been called to the attention of other members of this local committee, but apparently nothing was done to remedy it. If buckling should be found to have actually occurred the applicability of the test to actual dam design will be extremely limited.

In conclusion, the writer wishes to quote Professor Cain's closing remarks‡ as they so exactly and so aptly express his own attitude toward the whole subject:

"The author has written very convincingly of the serious effect of water-soaking in an arch dam. There can be no doubt of its influence, of the deformation of the foundation, and of the variation in E. He seems well aware that an exact solution of the arch dam is not to be looked for, so that all an engineer can do is to examine the various influences and combine them to effect a practical solution."

The writer wishes to thank those who have contributed to the discussion of his paper, especially Professor Cain, Dr. Vogt, and Mr. Paaswell, who have given liberally of their time and effort and by their discussions have added very materially to its value.

* "Corona Tests at High Altitude," *Transactions*, Am. Inst. Elec. Engrs., Vol. XXXVII, Pt. I (1918), p. 91.

† "Drang und Zwang," by A. Föppl and L. Föppl, Bd. I, Second Edition, p. 94.

‡ *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 277.

INCREASING THE EFFICIENCY OF PASSENGER TRANSPORTATION IN CITY STREETS

Discussion*

By John A. Miller, Jr., Assoc. M. Am. Soc. C. E.†

John A. Miller, Jr.,‡ Assoc. M. Am. Soc. C. E. (by letter).§—Elimination of parking on Fifth Avenue, New York City, during the recent subway strike resulted in an increase of more than 50% in the speed of vehicular movement between 32d and 52d Streets. On June 17 the writer found by careful investigation that the average speed of north-bound private automobiles in this area between 3:00 p. m. and 6:00 p. m. was about 4.3 miles per hour. On July 28, with anti-parking regulations in effect, a second check showed an average speed of approximately 7.2 miles per hour. Other conditions on these two dates were so similar that the elimination of parking must receive the major part of the credit for the improvement.

In previous discussion it has been stated by Messrs. J. C. Stevens,‖ J. P. Snow, T. T. McCrosky, and F. Lavis,¶ that the capacity of streets is determined by the intersections. That theory is only partly true. Of course the elimination of all crossings at grade would greatly increase the capacity of any street; but such elimination is a dream of the future. For the present cross movements are unavoidable, and the problem is to move the greatest possible amount of traffic during the period that the intersection is open.

If vehicles move across an intersection in six lines instead of four, the number crossing in a given time is increased 50 per cent. That was what happened on Fifth Avenue when parking was eliminated. On the day when the second of the two observations was made, north-bound vehicles crossed the 42d Street intersection in three lines, while on the first day they crossed in only two lines. The same space was available at the intersection on both occasions, but on the earlier date automobiles parked along the curb between 42d and 43d Streets created a bottle-neck on the far side of the crossing.

Undoubtedly much can be done to improve traffic conditions at street intersections. The full width of the roadway should be available for moving traffic. Left-hand turns prevent the most efficient utilization of space at an intersection and should be prohibited at heavy traffic points. This was brought out by Mr. Slattery.** Cutting back the corners to permit vehicles to turn

* Discussion on the paper by John A. Miller, Jr., Assoc. M. Am. Soc. C. E., continued from December, 1926, *Proceedings*.

† Author's closure.

‡ Associate Editor, *Electric Railway Journal*, New York, N. Y.

§ Received by the Secretary, December 11, 1926.

‖ *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1444.

¶ *Loc. cit.*, December, 1926, Papers and Discussions, pp. 2003, *et seq.*

** *Loc. cit.*, p. 1462.

on a longer radius while yet hugging the curb, as suggested by Mr. McCrosky, might prove advantageous in many places. Better control of pedestrian movement certainly would help. All this, however, will do little good if the roadway space between intersections is not used to the best advantage—a condition that does not now exist. Detailed consideration of these matters, however, lies outside the scope of this paper.

The argument has been advanced by Mr. Stevens that elimination of parking will not create an additional traffic lane because a certain number of vehicles will always be stopping along the curb to pick up or discharge passengers. This argument assumes that these vehicles are in the habit of drawing up close to the curb, but such is not the case. Often a solid line of parked automobiles renders the curb inaccessible. Consequently, vehicles picking up or discharging passengers stop in a second line in the middle of the roadway. Thus, the elimination of parking will actually create an additional lane for moving traffic despite the necessary stopping of occasional vehicles.

It cannot be denied that there is some curtailment of the rights of the individual when parking is restricted, but Mr. Long* goes rather far in saying that this is contrary to the American spirit. Restriction of individual rights for the common good has been practiced since the beginning of civilization. The "no parking" rule is only a new example of a well-established principle. Most cities have forbidden sidewalk obstructions. As Mr. Johnson† points out, to-day's protest becomes absurd to-morrow. If more people benefit by regulation than suffer thereby, it is clearly justified. Of this the public and not the individual must be the judge.

From all available evidence it appears that the beneficiaries of parking restriction would far outnumber the sufferers. A careless assumption often is made that nearly every one has an automobile and wants to park. The fact is that only a small fraction of the users of the public streets ride in private automobiles. Investigations made by the writer concerning traffic conditions on Fifth Avenue, New York, showed six times as many passengers riding in public transportation vehicles as rode in private automobiles. Traffic checks made elsewhere show similar conditions. Even in Detroit, Mich., the greatest automobile manufacturing center in the world, more than three-quarters of the users of the street ride in public transportation vehicles. Figures from a few typical cities are given in Table 6.

An even smaller proportion of private automobilists is shown by investigations made to determine the means of transportation used by customers to reach retail stores. Of 426 688 customers checked at 71 stores in various cities only 15.3% came by private automobile. Results of these counts are shown in Table 7.

Not all the users of private automobiles are "parkers". Many are en route and do not wish to stop. Possibly this is why the store counts show a smaller percentage of automobilists than the street counts. Many others leave their

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1451.
† *Loc. cit.*, p. 1460.

automobiles in garages. It is difficult to tell exactly what proportion of the users of private automobiles actually are "parkers". Figures on this phase developed by the four Chicago department stores mentioned in Table 7 show 452 automobiles parked free in public streets, as compared with 369 using paid storage space. It is safe to say that not one person in ten really profits by the parking privilege.

TABLE 6.—PASSENGERS CARRIED BY PUBLIC AND PRIVATE VEHICLES.

Location.	Number of persons using public transportation vehicles.	Number of persons using private automobiles.	Percentage by private automobile.
Fifth Avenue, New York, N. Y.	19 224	3 220	13.9
Broad and Market Streets, Newark, N. J.	38 000	6 000	13.6
Business district, Detroit, Mich.	57 150	18 281	24.1
Business district, Baltimore, Md.	58 859	12 509	18.1
Business district, St. Louis, Mo.	69 768	17 601	20.1

TABLE 7.—MEANS OF TRANSPORTATION USED BY STORE CUSTOMERS, PEDESTRIANS NOT INCLUDED.

City.	Number of stores.	Number of persons using public transportation vehicles.	Number of persons using private automobiles.	Percentage by private automobile.
New York, N. Y.	13	34 439	813	2.3
Chicago, Ill.	4	13 549	1 680	11.0*
Detroit, Mich.	27	158 801	36 272	19.1
Cleveland, Ohio	22	69 419	20 313	22.7
Baltimore, Md.	3	38 433	1 712	4.3
Brooklyn, N. Y.	1	28 800	2 640	8.4
Los Angeles, Calif.	1	17 759	2 058	10.4
Total	71	361 200	65 488	15.3

* Customers questioned in this investigation were specially selected as being of a type likely to have come by automobile.

No one has suggested that elimination of automobile parking on important streets offers a complete solution of the problem of traffic congestion; but experience has shown conclusively that it does afford relief. It can be done at once. The cost is practically nothing. A large majority of the public will benefit by the greater amount of roadway space available for moving traffic.

STRAIGHT LINE PLOTTING OF SKEW FREQUENCY DATA

Discussion*

By L. Standish Hall, Assoc. M. Am. Soc. C. E.

L. Standish Hall,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author has made a valuable contribution to the subject of skew frequency curves as applied to engineering data. He deserves especial credit for having developed an entirely original method of treating such curves. A study of the examples given leaves no room for doubt that the method presented will yield satisfactory results as far as determining curves that will fit a given set of data is concerned.

It is the writer's opinion that many of the variations in the types of frequency curves encountered in run-off data are possibly of an accidental nature and are due to the shortness of the records available, or what statisticians term "fluctuations of sampling". It would appear that if some system of frequency curves is adopted, having a sound mathematical foundation, such curves could be used as a means of comparing and correlating the various data. In this way the data from many streams could be compared and more satisfactory results would be possible than could be reached from the uncorrelated study of individual streams.

For this reason the writer is inclined to favor the Pearsonian frequency curves despite the difficulties of their application to hydraulic data. These difficulties are not due to any fault of Professor Pearson's theory, but are largely caused by the shortness of the records which must be used in applying the curves. The variation in the *c s* (attention to which has been called by the author) is merely an indication that the data used are of insufficient length to determine accurately the frequencies to be expected. The omission of this coefficient from the computations does not alter this condition, which is inherent in the data.

One important feature favoring the use of the *c v* and the *c s* is the fact that for a given type of curve, data having the same *c s* will have variations from the mean proportional to the *c v*.§ This fact is especially important in the correlation of data from different streams.

Furthermore, a study of the *c s* and *c v* of streams having long records, indicates that a more or less definite relation exists between these two coefficients.‖ It is possible that with the extension of stream-flow records, the correlation between these coefficients can be brought within narrower limits.

It appears essential to the writer that some measure be had of the reliability of the frequency curves drawn either by the author's method, or by any

* Discussion on the paper by R. D. Goodrich, M. Am. Soc. C. E., continued from January, 1927, *Proceedings*.

† Chf. Hydrographer, East Bay Municipal Utility Dist., Oakland, Calif.

‡ Received by the Secretary, November 13, 1926.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 162 and 196.

‖ *Loc. cit.*, Vol. LXXXIV (1921), pp. 248–249.

other method. Such a measure can be had.* If n is the number of terms in a series, p, the probability of an event occurring, and q, the probability of the event failing, the probable error of the event occurring is $e = 0.675 \sqrt{n p q}$. The probable error reduced to percentage of time would be, $e_t = \frac{e}{n} \times 100$. In Table 24 are given values of e_t for varying values of p, q, and n.

TABLE 24.

p.	q.	$0.675 \sqrt{p q}$	Values of e_t.			
			$n = 10$.	$n = 25$.	$n = 50$.	$n = 100$.
0.01	0.99	0.067	2.1	1.3	0.9	0.6
0.05	0.95	0.147	4.6	2.9	2.1	1.4
0.10	0.90	0.203	6.4	4.1	2.9	2.0
0.20	0.80	0.270	8.5	5.4	3.8	2.7
0.30	0.70	0.309	9.8	6.2	4.4	3.1
0.40	0.60	0.331	10.5	6.6	4.7	3.3
0.50	0.50	0.337	10.7	6.7	4.8	3.4

Elderton states:

"The general rule followed by statisticians when considering probable errors is that unless a result exceeds the expected by two or three times the probable error, it is not safe to assume that the particular case differs from the expected result."

To apply the results in Table 24 to a specific case, assume that the frequency curve drawn from a 10-year record shows that a run-off of 40% of the mean, or less, is to be expected 10% of the time. The probable error shows that in 50% of the cases, the run-off as determined would occur between 3.6 and 16.4% of the time. Considering the extreme case of three times the probable error, the run-off might never occur, or it might occur as often as once in 3½ years. If the curve had been determined from a 100-year record the probable error would be reduced from 6.4 to 2.0 per cent. Applying this to the case just considered, the run-off of 40% of the mean, or less, would probably occur between 8 and 12% of the time, or in the extreme case between 4 and 16% of the time. Thus, Table 24 should permit the engineer to determine the limits within which his frequency curve may be expected to vary.

The writer is greatly interested in the curves for the Wimmera River (Fig. 11†) as indicating the flexibility of the author's general equation. It would be of interest if the author would explain how the values of the five constants in this equation were determined.

Perhaps the most important application of probability methods is the one first made by Allen Hazen,‡ M. Am. Soc. C. E. It is believed that, by the use of recent developments in the methods of treating skew frequency curves, improvements in the application of probability methods to storage could be made and much more reliable results secured.

* "Frequency Curves and Correlation," by W. Palin Elderton, p. 132 *et seq.*

† *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1090.

‡ "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply," *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

DISTRIBUTION OF REINFORCING STEEL IN CONCRETE BEAMS AND SLABS

Discussion*

By Messrs. L. J. Larson and Anton Brandtzaeg.

L. J. Larson,† Assoc. M. Am. Soc. C. E. (by letter).‡—The problem of properly distributing the steel in a concrete beam is well worthy of consideration. The difficulty of designing a rectangular beam to carry the relative large moments at the supports when a T-section is available to carry the smaller moments at the center of the span, makes any logical method of reducing the moments at the supports especially welcome. The author's procedure is indeed convenient and if the partial continuity of a beam is due to some freedom of rotation at its ends his conclusions are justified.

In many beams, however, the ends are fixed and the tangents to the elastic curves remain horizontal at those points. Such is certainly the case for the types of beams given by the author as fixed.

If the ends of the beams are fixed the division of moment between the center and the end of the span depends on the variation of the moment of inertia of the beam and not on assumptions. The location of the points of inflection for various distributions of the moments is as indicated in the author's Fig. 2.§ However, the location of the points of inflection and the true distribution of the moment depend on the variation of the moment of inertia of the beam. Let I_s be the moment of inertia of the beam for the section from support to the point of inflection and let I_c be the moment of inertia of the central part of the beam, then the ratio, $\frac{I_s}{I_c}$, required to produce points of inflection at a distance, $k\,l$, from the support is shown in Table 3. The values of k lie between 0 and 0.5. The moment coefficients at the supports and at the center of the span are also given.

In Fig. 19 the ratio, $\frac{I_s}{I_c}$, and the moment coefficients are plotted against the values of k. For a rectangular beam having the same amount of steel at the center as at the supports the ratio, $\frac{I_s}{I_c}$, equals 1.0, and the moment coefficients agree with the accepted values for a fixed beam. For beam and

* Discussion of the paper by Boyd S. Myers, M. Am. Soc. C. E., continued from January, 1927, *Proceedings*.

† Research Engr., A. O. Smith Corporation, Milwaukee, Wis.

‡ Received by the Secretary, November 29, 1926.

§ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1108.

slab construction the center is a T-beam and the ends are rectangular beams. The moment of inertia of the center is greater than that of the end, and the bending moment at the ends of the beam will be reduced below the usual value for a fixed beam.

TABLE 3.

	Values of k.							
	0	0.10	0.15	0.20	0.25	0.30	0.40	0.50
Ratio, $\frac{I_s}{I_c}$	0	0.103	0.315	0.817	2.00	5.12	271	.....
Moment at support	0	0.045	0.064	0.080	0.093	0.105	0.120	0.125
Moment at center	0.125	0.080	0.061	0.045	0.032	0.020	0.005	0

To what extent the moments and stresses will be affected by the variation in the moment of inertia can best be shown by a specific example. Consider a T-beam having a flange 50 in. wide and 4 in. thick and a stem 10 in. wide. The total depth of beam and slab is 22 in. (to the steel). Assume 4 sq. in. of steel. The moment of inertia of such a beam is 18 570 in.^{4} At the support consider a rectangular beam 10 in. wide and 22 in. deep (to steel) with the same amount of steel. The moment of inertia of this beam is 11 550 in.^{4}

Using the notation in Fig. 19, $\frac{I_s}{I_c} = 0.62$, the bending moment at the support is $0.076\, w\, l^2$, that at the center of the beam is $0.049\, w\, l^2$, and the resulting stresses are $0.000071\, w\, l^2$ and $0.000041\, w\, l^2$ at the support and at the center of the beam, respectively. Thus, the stress at the support is about 70% higher than at the center of the beam if the areas of tension steel at the two points are equal. If the area of the steel at the support is reduced to 2.0 sq. in., or one-half that at the center of the beam, the ratio, $\frac{I_s}{I_c}$ is 0.40, the moment at the support equals $0.07\, w\, l^2$, the moment at center of the span equals $0.055\, w\, l^2$, the stress at the support is $0.000122\, w\, l^2$, and the stress at the center is $0.000046\, w\, l^2$. Hence, the steel at the support will be stressed about two and one-half times as high as that at the center of the span. Any further reduction of the steel at the support aggravates the situation. Simply reducing the steel at the support does not reduce the moment a corresponding amount.

If the amount of the steel at the support is much below that required, it must be over-stressed, or the steel at the center of the span will be subjected to less than the working stress. Of course, a re-adjustment will take place before the structure collapses. Most statically indeterminate structures have the happy faculty of shifting the load from the weak or over-stressed parts to those less stressed. However, this re-adjustment cannot take place until

some failure or yielding occurs. In the case of a concrete beam the concrete may flow, the steel may pass the yield point, or the rods may slip at the supports. One of these failures must occur before the center of the beam can receive a greater proportion of the moment. To what extent such a yielding at the supports may be tolerated, must be determined from practical consideration.

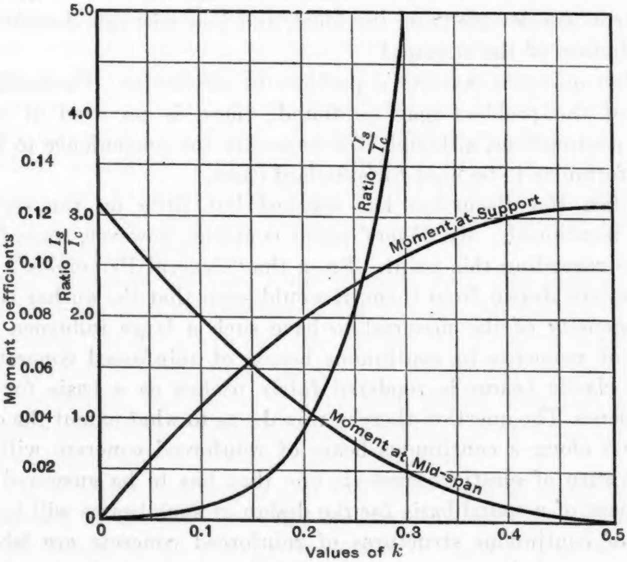

FIG. 19.

Shear Reinforcement.—Although it may be more convenient to use vertical stirrups as shear reinforcement, it is possible to make bent-up bars quite effective. Since some bars are generally bent up in continuous beams to help carry the negative moment they may very well be used as shear reinforcement. Some tests made by the writer at the University of Illinois show the effectiveness of such bars.

The beams tested were 8 in. wide, 17 in. deep, and 18 ft. long; d was 16 in. They were supported on a span of 12 ft., with an overhang of 3 ft. at each end. Equal loads were placed at the third points of the central span and 2 ft. 8 in. outside each support. This loading produced a moment at the supports twice that at the middle of the beam. About 2% of steel was provided at the supports. The bars were bent up at angles of 22½, 30, and 45° at various spacings up to 16 in. The distance of the first bend from the support varied from 8 to 24 in. In all, forty beams were tested. Of this series practically all the beams developed average vertical shearing stresses of more than 400 lb. per sq. in. and the failure was seldom caused by diagonal tension. Judging from these tests it appears that in beams having bent-up bars the amount of stirrup steel may often be materially reduced if such bars are considered as shear reinforcement.

ANTON BRANDTZAEG,* ASSOC. M. AM. SOC. C. E. (by letter).†—The distribution of the steel should agree with the distribution of the stresses. When taken in its broadest aspects, the determination of the stresses set up in an actual structure by any given condition of loading, usually should involve the consideration of two different questions: (1) What stresses would exist if the same structure were built of an ideal, homogeneous, isotropic, and elastic material; and (2) in what manner does the material actually to be used for the structure deviate from the ideal, and how will this deviation influence the distribution of the stresses?

The first question involves a problem in mechanics. Generally a definite solution of the problem may be found; there is no need of resorting to arbitrary assumptions, although it is necessary for convenience to have simple, standard formulas to be used for standard cases.

Thus far, the discussion has touched but little on the second of the questions mentioned. Mr. Myers' paper contains, however, some far-reaching statements regarding this point. From the "General Principles"‡ stated, and the conclusions drawn from them, it would seem that the author considers the non-homogeneity of the material to have such a large influence on the distribution of moments in continuous beams of reinforced concrete, that the theory of elastic beams is rendered fairly useless as a basis for computing these moments. The question thereby raised—as to what extent the distribution of moments along a continuous beam of reinforced concrete will follow the laws of flexure of elastic beams—is one that has to be answered before the establishment of a sound basis for the design of such beams will be completed.

Tests of continuous structures of reinforced concrete are laborious and expensive; nevertheless some reports of such tests are to be found in current engineering literature. It may not be out of place in this discussion to refer briefly to a few such tests, as they throw some light on the problem in general, as well as on some of the particular recommendations contained in the paper.

Among tests made in the United States, those carried out by Dr. Mikishi Abe at the University of Illinois on a large number of rigid frames of reinforced concrete, are well known.§ The general result of the tests was that a fair agreement was found between the stresses computed on the assumption of elasticity and those found in the tests by strain-gauge measurements.

Among tests made in Germany there are some which attack in a very direct way the problem of moments in continuous beams. As early as 1907 the construction firm Wayss and Freytag, Incorporated, undertook the testing of three large T-beams, continuous over two equal spans of about 19 ft. 4 in. clear width.‖ Naturally the method used in these early tests was not far enough developed to give very complete results, and the tests now have mainly

* Asst. Engr., Spanish River Pulp & Paper Mills, Ltd., Espanola, Ont., Canada.

† Received by the Secretary, December 2, 1926.

‡ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1106.

§ "Analysis and Tests of Rigidly Connected Reinforced Concrete Frames," by Dr. Mikishi Abe, *Bulletin No. 107*, Univ. of Illinois, 1918.

‖ Professor E. Moersch, who planned the tests, describes them in his book, "Der Eisenbetonbau." Fifth Edition, Stuttgart, 1920, Vol. 1, second half, pp. 318*ff*.

historical interest, although they demonstrated that the beams must have acted very much like continuous beams of elastic material.

A comprehensive series of tests of continuous T-beams was made by Scheit and Probst, first reported in 1912.* Professor Probst describes the tests in his book on reinforced concrete.† A description of the tests is also given in a recent book by Taylor, Thompson and Smulski;‡ a general description of the tests is therefore hardly warranted.

The test data give detailed information on the loads applied, the deflection of the beams, and the rotation of the beams over the supports. Professor Probst compares the relative deflections of the beams under several conditions of loading and support with those computed for elastic beams of uniform cross-section under the same conditions. He finds fair agreement, and concludes that continuous beams of reinforced concrete, freely supported, in general follow the same laws as similar elastic beams. He finds that for beams poured monolithically with the columns, the effect of the columns on the bending moments is very pronounced.

Few strain measurements were made in these tests, and little is known of the internal stresses actually existing in the beams. All the beams failed by tension in the steel, and a rough estimate of the stresses at failure is made on the basis of the elastic strength of the steel as determined by auxiliary tests. As the steel evidently was not of a very mild grade, even this estimate is rendered rather uncertain. Nevertheless, a sample of the results may be worth mentioning in connection with the present discussion.

Two identical beams, continuous over two spans, both of which were loaded, failed by tension in the steel over the support. An estimate on the basis of the yield-point stress for the steel gives a negative moment over the support at failure of slightly less than $\frac{W\,l}{10}$. For a similar elastic beam of uniform cross-section the moment would be nearly $\frac{W\,l}{8}$. (The load was only approximately uniformly distributed.) These tests indicated, therefore, that the moment over the support was less than that computed for elastic beams. It is interesting to consider this beam in relation to the views expressed by the author. The beam had the same area of steel over the support and at the center of the spans. Suppose that the designer of such a beam had decided to assume the moments at the center of the span and at the support to be the same, and had used as the governing condition that the sum of the positive and the average of the negative moments should be $\frac{W\,l}{8}$. He would then have distributed the steel as it was in this beam and would have designed it for $\frac{W\,l}{12}$ at the support.

* "Untersuchungen an durchlaufenden Eisenbetonkonstruktionen," by H. Scheit and E. Probst, pub. by J. Springer, Berlin, 1912.

† "Vorlesungen über Eisenbeton," by Professor Dr. E. Probst, pub. by J. Springer, Berlin, 1917, Vol. 1, pp. 442 to 495.

‡ "Concrete Plain and Reinforced," by F. W. Taylor, S. E. Thompson, and E. Smulski, John Wiley & Sons, N. Y., 1925, Vol. 1, pp. 64 to 68.

Actually, the steel stress corresponded to a negative moment of about $\frac{W l}{10}$, and the beam would have been about 15% too weak for the load for which it was designed. The author's recommendation for beams over end spans is, however, that they should be designed for $\frac{W l}{10}$ at the center and $\frac{W l}{20}$ at the interior support.* The beams referred to had twice as much steel at the support as recommended by the author; the fact that they failed at that point and not in the center must indicate that they had too little, rather than too much, steel there.

The author's statement that "an interior beam framing into a column at each end, in line with similar beams in the adjacent spans, unquestionably is fixed at its supports", does not agree with the results from some tests of beams over three equal spans, resting on concrete columns, which were included in this series. The two end spans only were loaded. Large rotations were observed at the interior supports, and large cracks developed in the top of the beam along the unloaded middle span. It may also be noted that very large cracks developed in the columns themselves.

Another series of tests of considerable interest is that carried out by Bach and Graf in Stuttgart, reported in 1920.† The tests were made under the auspices of the German Committee on Reinforced Concrete. The plans for the tests were proposed by Professor E. Moersch, who has included a description of the tests in his book.‡

The specimens for these tests were of rectangular cross-section, in most cases they were about 11¾ in. wide and 7⅞ in. deep. The beams rested on two supports, about 9 ft. 10 in. apart; they were extended as cantilevers about 4 ft. past the supports, both ways in some tests, one way in others. Equal loads were applied at six points uniformly spaced between the supports; in addition, there was one load at each cantilever end of the beam, about 3 ft. 3⅜ in. outside the supports. These latter loads were applied by separate mechanisms, and were regulated throughout the tests so that the cross-sections of the beams immediately over the supports remained in their original vertical position. The beams thus were actually fixed at one or both of the supports; the magnitude of the cantilever loads required to maintain this condition gave direct measurements of the negative moments.

Three sets of tests, each including three identical specimens with cantilevers at both ends, will be mentioned here. In two of these sets the steel was distributed approximately according to the distribution of moments in an elastic beam of uniform cross-section, fixed at both ends. The quantities of steel in one set were 0.3% at the center and 0.63% over the supports; in the second set the percentages were 0.77 and 1.63, respectively. In both these sets the negative moment, as actually measured, agreed closely with that

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1112.

† "Versuche mit eingespannten Eisenbetonbalken," by Dr. Ing. C. Bach and O. Graf, Heft 45, Deutschen Ausschusses für Eisenbeton, Berlin, 1920.

‡ "Der Eisenbetonbau," by Professor E. Moersch, Fifth Edition, Vol. 1, second half, pp. 310–318.

computed for elastic beams, throughout all the stages of loading. The measured moment slightly exceeded the computed one at the higher loads. Cracks developed at about the same rate at the center and over the supports, and the final failure occurred through tension in the steel, in some cases at the center, in others at one of the supports. Professor Moersch takes this as an indication that the steel in these beams had been properly distributed.

In the third set of tests the distribution of the steel was the reverse of that in the second set, the quantity at the center being 1.63%, and that over the supports 0.77 per cent. A special interest is attached to this set of tests in connection with the present discussion, since the distribution was very nearly that advocated by the author for interior spans of continuous beams. At early loads the negative moment in this set, as in the previous ones, followed the values computed for similar elastic beams of uniform cross-section. After cracking had started over the supports, the observed negative moment was smaller than the computed one, the difference increasing as the test proceeded. However, the difference between the computed and the actual moment was not as great by far as if the moments had been distributed proportionately with the steel. To take an example: At a total load between supports of about 22 000 lb. the negative moment for a similar fixed elastic beam of uniform cross-section would be about 18 300 ft-lb. The moment measured was about 14 400 ft-lb., or 79% of the "theoretical" value. Taking the moment at the center of a similar beam simply supported, and assuming it to be distributed between the center and the support proportionately with the steel, one computes a negative moment of about 8 700 ft-lb., or 47.5% of the "theoretical" value. The steel stresses at this load, computed from the observed moments by ordinary formulas, amounted to about 16 500 lb. per sq. in. at the center and 40 000 lb. per sq. in. at the supports. The yield point for this steel was at about 43 000 lb. per sq. in. As the load was raised, the increase in the negative moment soon became very slow; the steel over the supports yielded, with cracks opening up, and the further increase in load had to be carried by the steel at the center. Ultimately, the beams of this set carried about the same load as those of the preceding one.

These tests show that the designer may distribute the steel according to the recommendations of the author only if he is prepared to accept the certainty of premature yielding of the steel over the supports, with the resulting large cracks. Until such yielding occurs the moments will not follow an arbitrary distribution of the steel, even if they do deviate somewhat from the "theoretical" values. Certainly the foregoing steel stresses do not indicate that the material in these beams had been distributed to the best advantage.

The tests here mentioned demonstrate the advisability of showing caution in basing the distribution of the steel in continuous beams on too loose assumptions. They indicate in a general way that a computation based on the laws of flexure of homogeneous beams of uniform cross-section is likely to give for the bending moments values that are reasonably close to those actually existing in the beams. They do not, however, point to any definite con-

clusions as to just in what manner the action of continuous beams of reinforced concrete deviates from that of homogeneous beams.

The great difference between a homogeneous beam and one of reinforced concrete in which "hair cracks" have developed on the tension side of the beam, is very obvious. From a general consideration of the case it might seem that, in order to obtain correct values of the moments, one should neglect the concrete on the tension side of the neutral axis in computing the moments of inertia, and introduce different moments of inertia, according to the different amounts of steel and shapes of the concrete section at the center and over the support, into the ordinary formulas for moments in continuous, elastic beams. In fact, other discussors have explicitly assumed this to be so. The tests mentioned do not seem to indicate that the moments computed in this manner will, on the whole, be more correct than those computed on the assumption that the moment of inertia is uniform throughout the beam. The beams over two equal spans, tested by Probst, had the same amount of steel at the center and over the support. The beams were of a T-section, with a slight amount of compressive steel at the support, and the ratio, $\frac{I_p}{I_n}$, for these beams was about 1.08, neglecting tension in the concrete. According to this, the negative moment should have been only slightly smaller than that computed for a beam of uniform cross-section. However, the test indicated a negative moment as much as 20% smaller. Most of the beams tested by Bach and Graf had twice as much steel at the supports as at the center. The ratio, $\frac{I_p}{I_n}$, for the second of the three sets of beams was about 0.61. Accordingly, the negative moment should have been about 10% higher than that computed for a beam of uniform cross-section. However, the negative moment observed exceeded only by about 3% the moment computed for such a beam. For the extreme case of the third set of beams, for which $\frac{I_p}{I_n} = 1.63$, by computation the negative moment should be 87% of that for a beam of uniform moment of inertia, while the observed moment was 79 per cent.

Professor Moersch,* gives an interesting discussion of the question as to what sections should be considered in computing the moments of inertia of concrete beams for the purpose of determining moments in continuous beams. The discussion is based on the results of a number of tests of simply supported beams, some of which had rectangular cross-section, while some were T-beams, and some had an "inverted T-section" with the steel at the same side of the beam as the slab. The latter beams represented the conditions at the supports of a continuous T-beam. In all these tests the relations between bending moments and deformations of a short length of beam were directly determined.

Professor Moersch applies these observed relations in a semi-analytical study of the action of several types of beams with one or both ends fixed

* "Der Eisenbetonbau," Fifth Edition, Vol. 1, second half, pp. 284–310.

He assumes in each case a beam in which the steel has been distributed according to the distribution of moments in a similar beam of uniform cross-section. The dimensions of the beam and the reinforcement have been chosen so that the cross-section at every point along the beam is equal to that of some one of the simple beams tested. By means of the observed relations between moments and deformations of short lengths of beam, he determines what total deformations of the assumed beams would correspond to the distribution of moments considered. He treats in this manner a rectangular beam fixed at both ends, a rectangular beam fixed at one end, and a T-beam fixed at one end. He finds in all cases that the total deformations of the beams, determined on this basis, agree very nearly with the conditions of support in each case; in other words, that the distribution of bending moments used is very nearly right. Professor Moersch draws from this the general conclusion that if the steel in a concrete beam is distributed in accordance with the bending moments computed for a similar elastic and homogeneous beam of cross-section equal to the full concrete section, then the actual bending moments will correspond very nearly to the moments thus computed. He refers to the tests by Bach and Graf, previously mentioned, as supporting this conclusion.

The tests seem to indicate that in many cases of great practical importance this statement by Moersch may be nearly right. It is obvious that if this is so, it is due to a rather complex set of reasons, and it would seem that there must be limitations to the applicability of the rule.

Among the possible sources of such limitations, there is one which was not considered in the analysis by Moersch, and which is hardly capable of being fully allowed for analytically, namely, the danger of slipping of bars over the supports of a continuous beam. The writer is convinced that serious limitations on the conformity of the actual bending moments in continuous beams of reinforced concrete with the computed ones, may arise in many cases from this source. There are two reasons why the danger of slipping of bars is probably greater at the supports of a continuous beam than at most other points of a reinforced concrete structure. In the first place, the maximum bond stresses occur at the supports. At the same time the steel is there stressed so highly that hair cracks in the concrete are almost certain to develop even at working loads. It was amply demonstrated by the tests made by Professor Abrams at the University of Illinois,* that when hair cracks develop along any part of a beam, the resulting "anti-stretch slip" impairs greatly the ability of the beam to resist the "regular" bond stresses along that part. In the second place, in most cases the steel in such a beam is firmly supported on the forms during the pouring of the concrete and is prevented from settling together with the concrete. As the settlement close to the top of the beam will be greater than that close to the bottom, the detrimental effect of the settlement on the bond strength will also be more pronounced at the top. That such settlement greatly affects the bond strength, was also shown by Professor

* "Test of Bond Between Concrete and Steel," by Duff A. Abrams, M. Am. Soc. C. E., *Bulletin No. 71*, Univ. of Illinois, 1913.

Abram's tests.* It should be noted in this connection that in the German tests, smaller bars were used than would ordinarily have been used in similar beams in American practice. Nevertheless, it may be that slipping of bars may have occurred and this may be responsible in part for the small value of the negative moment in the beams over two spans, as tested by Probst. The fact that cracks suddenly opened up at a small distance from the support some time before the maximum load was reached,† may possibly indicate that this was the case.

On the basis of the tests referred to it would seem that the designer can hardly do better than to compute the bending moments in a continuous beam of reinforced concrete as if he were dealing with an homogeneous beam of the same cross-section as the concrete beam. The moments thus computed are likely to be reasonably close to the actual ones. It is doubtful, at best, whether the introduction of varying moments of inertia according to the varying amounts of steel will give results that are more correct than those obtained by the simpler method. The designer should keep in mind, however, that he is dealing with a non-homogeneous material, and that in particular the bond between the concrete and the steel may in many cases be insufficient to maintain the conditions on which the computation is based. This consideration may often make it advisable to provide some extra steel at the center of the beam, in order that the beam may be strong enough there, even if some slipping of bars should occur at the supports.

The possibility that slipping of the bars may act to reduce the negative moment at the support, however, should not be considered as a reason for reducing the steel at the supports. That would only result in increasing, not only the tension in the steel, but also the bond stresses. The ultimate result of any large reduction of the steel at the supports from the amount required for the computed moment, will be premature yielding of the steel, or bad slipping, either of which will cause cracks to open. It will be seen that the danger of slipping of the bars over the supports furnishes one reason, in addition to the necessity of providing for an unequal distribution of loading, why the sum of the positive and the negative bending moments as given in the standard formulas of building codes, should be greater than $\frac{W\,l}{8}$.

There seems to be need of further experimental study of the question as to how the distribution of moments in continuous beams of reinforced concrete deviates from that in homogeneous elastic beams. Such experiments would doubtless be of the greatest value if they could give information not only as to the deviations found in certain cases, but also as to the causes underlying these deviations.

* "Test of Bond Between Concrete and Steel," by Duff A. Abrams, M. Am. Soc. C. E., *Bulletin No. 71,* Univ. of Illinois, 1913, p. 105.

† "Vorlesungen über Eisenbeton," by E. Probst, 1917, Vol. 1, p. 459.

PRODUCING CONCRETE OF UNIFORM QUALITY

Discussion*

By Thomas K. A. Hendrick, Assoc. M. Am. Soc. C. E.

Thomas K. A. Hendrick,† Assoc. M. Am. Soc. C. E.—It is the understanding of the speaker that the Hydro-Electric Power Commission of Ontario has used, during the course of the installation of about 640 000 cu. yd. of concrete under scientific control, coarse aggregate of every kind, including gravel and broken stone.

Under the general heading of "Selection of Materials",‡ Mr. Young states that,

"* * * although control can be obtained when the materials vary, it is obtained more easily and cheaply when they are uniform. For this reason it is desirable to limit the supply of each to one source and to choose those known to vary least, even when this involves some additional expense."

In the use of a local stone for the coarse aggregate, which is apparently of poorer stuff, the expense of transportation of the apparently better coarse aggregate is overcome. Since in many cases to secure the required strength it would be necessary to use more cement, of course the use of the local material is not always a clear economic gain even if transportation expense is avoided.

Experiments performed by the writer§ showed that, in some instances, as in the case of the Minnesota Kettle River sandstone which pulverizes easily under the pressure of the thumb and forefinger, a concrete may result which is higher in strength than that made with trap-rock, gravel, or granite. The same general conclusion has been reached independently.‖

It has not been demonstrated that the densest concrete gives the highest strength. Repeated experience has shown it to be highly desirable to experiment on each local material which has any potential suitability as a coarse aggregate. If the author would limit the supply to one source, it should be stipulated that many trial comparative experiments be performed on every available local stone before a decision to limit to the one source be reached. To secure uniformity is the desideratum; but uniformity can be secured by combination of methods rather than at the sacrifice of a local material without fair trial.

* Discussion of the paper by Roderick B. Young, Esq., continued from January, 1927, *Proceedings*.

† Civ. Engr., New Rochelle, N. Y.

‡ *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1394.

§ "A Study and Determination of the Relative Efficiency of Sixteen Different Kinds of Broken Stone and Two Different Kinds of Gravel When Used as Aggregate in Concrete," *Engineering News*, March 9, 1916.

‖ "An Investigation of the Concrete Road Making Properties of Minnesota Stone and Gravel," by Prof. Charles Franklin Shoop, *Studies No. 2*, Univ. of Minnesota, 1915.

WATER-PROOF MASONRY DAMS

Discussion*

By Messrs. Edward Wegmann, Malcolm Elliott, and K. E. Hilgard.

Edward Wegmann,† M. Am. Soc. C. E.—The author proposes to make uplift in a masonry dam impossible, by placing in the body of the dam, near its up-stream face, a water-tight membrane of metal, asphalt, or some other suitable material, and by carrying this membrane down to an impervious foundation.

In addition, he recommends the somewhat customary system of under-drainage, to ensure the removal of any seepage from the foundation bed, which he assumes to be practically water-tight. If this should not be the case, the drainage system might cause serious leakage, but the author asks: Why build on such a foundation? If the foundation is not entirely water-tight, he recommends the usual cut-off wall, with a water-proofing membrane within it. To what depth the cut-off wall should be carried down, he does not state, but it is evident that the value of his system, if found to be perfectly practical, would only apply to cases where a water-tight foundation could be had at a reasonable depth.

The water-tight membrane would, of course, have to be tied to the remaining part of the dam by rods, etc., so as to enable the whole mass of masonry to act as a unit, and to prevent all possible leakage past the membrane. If this can be done successfully, the additional strength often given to masonry dams to prevent uplift can be greatly reduced and, possibly, omitted entirely.

There are a number of practical difficulties in placing a water-tight membrane in the body of a dam, as fully discussed by others. After all, the only way of determining positively the value of the water-proofing system proposed by the author is by actually building a dam with a water-proof membrane. It is to be hoped that in the near future he may be able to build such a dam.

The paper re-opens a question much discussed by engineers about twenty years ago, and about which there is still quite a difference of opinion, namely, does uplift exist in an ordinary masonry dam, built on a good, fairly water-tight foundation? and if it does exist, how much additional strength should be given a dam, to enable it to resist such upward pressure?

Prior to 1895, there was not a case on record in which the failure of a masonry dam had been attributed partly, if not entirely, to upward pressure. Distinguished foreign and American engineers had written memoirs and books, discussing the proper manner of designing masonry dams, but none

* Discussion of the paper by W. Watters Pagon, M. Am. Soc. C. E., continued from January, 1927, *Proceedings*.

† Cons. Engr., New York, N. Y.

of them, as far as the speaker knows, had mentioned the fact that with a bad, pervious foundation, uplift under the base of a dam might exist. Moreover, these engineers had designed and actually built about thirty masonry dams, in different parts of the world, without paying any attention to a possible uplift under the bases or to ice pressures. These dams range in height from 50 to 164 ft., and some of them are now more than 60 years old. None of them has failed.

On April 27, 1895, the Bouzey Dam, built in 1878-81 near Epinal, France, failed, causing a great loss of life and property. This dam was founded on red sandstone, which was much fissured and quite pervious. Much trouble had been experienced in the foundation trench from springs, and to prevent leakage under the dam a cut-off wall, 2 m. thick, was built at the up-stream face. The failure of this dam has been attributed to a greater tension at the up-stream face than the masonry could withstand, this pressure being, probably, increased by uplift.

In December of the same year, the late J. D. Van Buren, M. Am. Soc. C. E., read before the Society a paper,* in which he recommended that all high masonry dams should be designed for a full uplift—due to the whole head in the reservoir—acting on the base and on each assumed joint above it, and, also, in northern latitudes, for an ice pressure of 40 000 lb. per lin. ft., acting at the crest of the dam. Mr. Van Buren submitted a plan for a dam, designed according to his principles; for a height of 250 ft. it had a base of 352 ft.

Considering the fact that of all the masonry dams built, including a number in which a large amount of uplift and ice pressure were considered in the design, none has a height equal to its base, the folly of taking such extreme assumptions is evident.

There was much discussion among engineers about Mr. Van Buren's paper. It was pointed out that according to his assumption, the dam would be floating, and that while with a badly fissured foundation, there might be considerable uplift at seams and fissures, it was not logical to assume this uplift as extending over the whole base. It was probably limited to a certain percentage of the foundation, and certainly did not extend under the whole base with the full head that was in the reservoir.

The construction of the New Croton Dam, the height of which was to exceed that of all existing dams by more than 100 ft., had been begun in September, 1892, and it was an important question for the late Alphonse Fteley, Past-President, Am. Soc. C. E., and Chief Engineer of the Aqueduct Commission of New York, under whose direction the dam was to be built, to determine whether, in view of Mr. Van Buren's paper and the discussion thereon, he should modify the profile of the New Croton Dam to include an allowance for uplift and for possible ice pressure.

The speaker was Mr. Fteley's Assistant in the design of the New Croton Dam, and is, therefore, familiar with the reasons which influenced his decision.

* "Notes on High Masonry Dams," *Transactions*, Am. Soc. C. E., Vol. XXXIV (1895), p. 493.

The four dams given in Table 1 had, up to that time, been constructed on the Croton water-shed, the last three being designed and built under the supervision of Mr. Fteley.

TABLE 1.—DAMS ON CROTON WATER-SHED PRIOR TO 1892.

Name.	Built.	Maximum height above lowest point of foundation, in feet.
Boyd's Corners	1866-72	70
Sodom	1888-93	98
Titicus	1890-95	135
Carmel	1891-95	65

All these dams were founded on rock similar to that which was known to exist under the New Croton Dam. Mr. Fteley was of the opinion that any uplift which might exist under any of these dams, and which might occur under the New Croton Dam, would be limited to a small percentage of the area of the base. As regards ice pressure, more than a foot of ice had formed in the reservoirs of the dams given in Table 1, without causing any trouble. Mr. Fteley thought that any sheet of ice that might form in the New Croton Reservoir would buckle before it could exert a great pressure on the dam.

For these reasons, he decided to build the New Croton Dam without any allowance for uplift or ice pressure. This dam, which has a maximum height of 297 ft., was completed in 1907, and to date has not shown the slightest sign of any weakness.

The next case of the failure of a large masonry dam due to poor foundation, undermining, and uplift occurred at Austin, Tex., on April 7, 1900, when the dam built across the Colorado River, about two miles above the city, was ruptured. This dam had been built in 1891-92. The water-shed above the dam contains about 50 000 sq. miles, consisting largely of mountainous country. For two or three days, a very heavy rainfall occurred in this water-shed, causing the river to rise until it flowed with a depth of more than 11 ft. over the dam. The dam broke, the current found its way through the gap and shoved two sections of the dam, each about 250 ft. long, bodily about 60 ft. down stream, without the slightest overturning, leaving them almost parallel with their original positions. During the night these sections were destroyed by the stream.

In this case, the dam had been built on limestone, which was in places so soft that it could be removed with picks and shovels. Moreover, it had been built directly over a geological fault, 75 ft. wide, full of adobe, with red streaks of clay, extending to an indefinite depth. The current passing over the dam had undermined the front toe. Under these circumstances there can be no question but that a large uplift was exerted under the dam.

In 1900-06 the late Frederic P. Stearns, Past-President, Am. Soc. C. E., and his assistants, designed and built the Wachusett Dam across the south

branch of the Nashua River, to form a large storage reservoir for the Metropolitan District of Boston, Mass. This dam has a maximum height of 228 ft. above the lowest point in the foundation. Its cross-sectional area for the same depth is considerably greater than that of the New Croton Dam.

Discussing with Mr. Stearns why he made so great a difference, the speaker was told that the City of Clinton, Mass., was about ½ mile below the dam, that great apprehension of a possible failure was felt by the residents; and Mr. Stearns added: "If I had not made the dam unusually thick, I would never have been able to get my plans accepted."

The basis on which this increased thickness was computed was by estimating an uplift under the whole base of the dam, beginning with two-thirds of the full head in the reservoir at the heel and diminishing uniformly to zero at the toe; and also by assuming an ice pressure of 47 000 lb. per lin. ft., acting at the crest of the dam. Under the circumstances, Mr. Stearns, no doubt, was justified in assuming a very large factor of safety for the Wachusett Dam.

Soon after the completion of this important dam, Mr. Stearns was appointed as one of the Consulting Engineers of the Board of Water Supply of the City of New York, under the direction of which the Ashokan, Kensico, and Gilboa Dams were built. In designing these dams the same allowance was made for uplift as in the Wachusett Dam, and a large ice pressure was assumed. As the City of Kingston was about 20 miles below the Ashokan Dam, and the City of White Plains only 3 miles below the Kensico Dam, and as practically unlimited financial means were available, all will agree that the engineers were justified in adopting for these dams unusually strong profiles.

Two more dams have been built in the Croton water-shed in recent years, namely,

	Built in.	Maximum height, in feet.
Cross River Dam............	1904-08	153
Croton Falls Dam............	1906-11	167

Both these dams were designed according to the principles adopted for the Wachusett Dam, the ice pressure being assumed at 24 000 lb. per lin. ft. for the Cross River Dam and at 30 000 lb. per lin. ft. for the Croton Falls Dam. Both these structures are located on affluents of the Croton Dam, about 15 miles up stream from the New Croton Dam, where there are few buildings and only a sparse population.

There exists, therefore, in the Croton water-shed this rather illogical condition: Five dams, including the New Croton Dam, the most important of all these structures, designed without taking uplift and ice pressure into account; and two dams designed by taking uplift and ice pressure into consideration, the latter having much larger profiles than the former, and, of course, involving a corresponding increase in cost. If the engineer had not to consider cost, his problems would be comparatively simple. By making the base of a dam equal to its height, he would be amply safe in all cases, even if uplift and ice pressure were to be considered. True skill in engineer-

ing consists, however, in obtaining the required safety with a minimum of expense.

There is a third case on record of a dam being ruptured by uplift. On September 30, 1911, the structure at Austin, Pa., failed, causing the loss of seventy-five lives and of much property. In this case the dam was built on a pervious foundation of shale, the excavation being only about 6 ft. deep, and no cut-off wall being provided. The first time the reservoir was allowed to be filled in part the dam was curved about 1 ft. down stream by the water pressure.

The engineer in charge of the work sought the advice of a consulting engineer, and was told that the only way to make the dam safe was either by building a good cut-off wall to an impervious stratum, or by constructing on the down-stream side of the dam a rock-fill dam, strong enough to keep the masonry dam in place. The engineer recommended that both these expedients be adopted and estimated that the cost might be $30 000, and possibly much more. He was promptly discharged and the President of the wood pulp and paper company for which the dam had been been built, who was what is often called "a practical man" took charge of the work. The reservoir was gradually filled. When the water reached the level of its spillway, the dam failed by sliding on its foundation, exactly as had been predicted by the consulting engineer. This failure was undoubtedly due to uplift.

To sum up the case as it stands at the present time:

More than forty masonry dams, in excess of 100 ft. in height, have been built in various parts of the world, and are still standing, without taking uplift and ice pressure into account.

Seven masonry dams, including the Hetch Hetchy Dam, have been built with large profiles, taking uplift and ice pressure into consideration.

Three masonry dams, all less than 70 ft. in height, have failed. All were built on poor, pervious foundations, and there can be no doubt that these failures were largely, if not entirely, due to uplift.

In conclusion, the speaker would state that whether or not uplift should be considered, is a question for the exercise of good judgment on the part of the engineer. It depends mainly on the character of the foundation. If it were as bad as that of the dam at Austin, Pa., the engineer might be justified in assuming a full upward pressure. In cases where large populations live below a dam and where great loss of life would occur in case of a failure, the engineer could be justified in providing a much larger factor of safety than would be required in ordinary cases.

As far as ice pressure is concerned, the speaker knows of only one case in which this force was the primary cause of the failure of a dam. At Minneapolis, Minn., in 1893-94, a mill company had built a dam of ashlar masonry, to form a pond for supplying power. The dam was 535 ft. long, 18 ft. high, 12 ft. wide at the base, and 5.25 ft. wide at the coping. A retaining wall, parallel with, and 350 ft. distant from, the dam formed the opposite side of the pond. In winter, ice 4 ft. thick covered the pond. During the week the pond was usually drawn down 2 ft., or more, to meet the demand

for power, but on Sundays the pond was allowed to rise to its normal level. This caused the sheet of ice to exert a toggle-joint pressure on the dam.

The dam stood successfully until the spring of 1899. In February of that year, it was noticed that the dam was slowly revolving around its down-stream toe, the top of the dam having moved 10 to 12 in. from its original position. There was no sliding.

The ice was cut out back of the dam, which returned slowly almost half way to its original position. On April 30, 1899, a section of the dam, about 170 ft. long, slid out. This failure is supposed to have been caused by water getting under the dam through some cracks which were opened when it was slightly revolved the preceding February. In this case there were peculiar circumstances: A sheet of ice, 4 ft. thick and 350 ft. long, abutting at the end against the dam, and at the other end against a retaining wall.

About ten years ago, the speaker was engaged by the Quebec Streams Commission to design a dam across the St. Maurice River, in a locality where ice, 6 ft. thick, formed every winter. The dam which was to have a height of 80 ft. was founded on Laurentian gneiss, which was as solid as granite, and entirely free from seams and fissures. No allowance was made in this case for uplift, but an ice pressure of 50 000 lb. per lin. ft. was assumed to act at the crest. The dam was built successfully and is still in use.

MALCOLM ELLIOTT,* M. AM. SOC. C. E. (by letter).†—Water-proofing the up-stream face of a dam as proposed by the author would in many cases lead to construction difficulties which might offset the advantages. Simplicity should be sought in the design so that the materials can be placed at the least possible expense. Especially is this true in the case of work that is frequently menaced by bad weather and floods. The proposed membranes might cause serious delays to the construction program, increasing the hazards due to floods and weather so as to offset any saving in material that might result from the water-proofing.

It is believed, however, that when first-class construction is sought, water-tightness should be carefully considered and obtained by any reasonable details of the design that will not unduly delay or increase the cost of the work. So far as stability is concerned, many cases may occur in which the most economical method of counteracting upward pressure would be to increase the weight of the dam rather than to attempt to exclude the water. In any case, there is an element of danger in assuming that the water-proofing elements will be entirely effective. While membranes as proposed would probably cut off a large part of the leakage, the assumption that all water, and thus all upward pressure, would be eliminated, is hardly a conservative one.

The most serious objection to the proposed method of water-profing is that it fails of its purpose at the most critical plane—the base of the dam. It is on this plane that the majority of failures may be expected. The author proposes to use a cut-off wall with a water-proofing membrane within

* Maj., Corps of Engrs., U. S. Army, New Orleans, La.

† Received by the Secretary, December 6, 1926.

it. Presumably the cut-off wall is to be carried down to intercept the upper surface of impervious rock; otherwise leakage would find its way under the cut-off wall and exert its upward pressure on all parts of the base. The excavation of a trench for this purpose will often impair the stability of a dam by breaking up strata of rock which, while allowing leakage would, if undisturbed, be capable of sustaining the dam. The excavation for the cut-off wall would usually be very expensive by reason of the extraordinary care needed to avoid shattering adjacent rock to the extent that it too would have to be removed, thus resulting in a general deepening of the foundation and, therefore, an increase in the height of the dam and the pressure acting on it.

To free a dam from the menace of upward pressure by water-proofing might thus result in a marked increase of cost, which could easily more than offset the savings claimed, even if it were certain that the stratum intercepted was absolutely impervious, but the number of dam sites that will afford rock warranting that much confidence is indeed small. The usual condition will be that in spite of all cut-off walls, however deep and well water-proofed, upward pressure on the base may be expected, due to the presence of seams capable of conducting the water under the wall and to all parts of the base. It seems far better to recognize this danger to the dam's stability and care for it by providing suitable sub-drainage and a sufficient weight of masonry to resist all forces.

The difficulties that would arise during construction due to the use of the proposed membrane should be mentioned. These difficulties include the support of the membrane during the pouring of concrete and the depositing and spading of concrete within the form, because of the area being broken up by the membrane and whatever appliances are used to support it. In the 3-ft. space up stream from the membrane the placing of concrete would be slow and tedious. Care would have to be taken that this space were not filled faster or slower than the remaining area within the form; otherwise, the pressure of the green concrete would displace the membrane, unless it were rigidly supported.

Concrete dams of considerable height are usually erected in sections from 30 to 40 ft. long and each section in "lifts" of not more than 12 ft. because of the difficulty and expense of installing forms that will support the pressure occurring when concrete is poured continuously to greater heights. This process results in approximately horizontal planes of weakness in each section, to which water may penetrate and on which sliding or overturning of the upper part of the structure is conceivable. It is easily possible, however, to treat these construction planes in such a way as to obviate any danger of failure. The surface obtained at the conclusion of a pour can be roughened by moulding troughs into which the concrete of the next lift above will be poured, and by scrubbing the new surface with wire brooms so as to remove laitance and expose portions of the aggregate; and bond may be obtained by driving reinforcing steel several feet down into the concrete leaving the upper ends protruding so as to hold down the next layer when

poured. All these precautions can be effected by the use of a fair-sized labor gang in each form for a few hours just after the pour is completed and without in any way interfering with the pouring of the concrete or delaying future construction, but they can not be depended on to prevent all leakage and the pressure resulting from it. However, leakage should not occur elsewhere than at the horizontal and vertical joints if the concrete is properly proportioned, mixed, and placed.

The writer was connected with the construction of a dam nearly 1 mile long and impounding the water to a height of about 95 ft. It was constructed in sections, each from 30 to 38 ft. long in the direction of the long dimension of the dam, and the blocks were built by pouring successive layers of concrete, usually 6½ ft. thick, one above the other. The top of each layer was roughened by moulding grooves therein into which the concrete of the next layer above protruded; the whole surface was scrubbed to remove laitance and expose portions of the stones composing the aggregate; and a thin layer of neat cement grout was applied before the next layer of concrete was placed. Notwithstanding these precautions, a number of these seams, probably less than one-quarter of the total number in the dam, allowed sufficient leakage to visibly moisten the down-stream face of the dam. In no case did the water issue in streams. The upward pressure produced by this leakage is unknown. It is conceivable that the water penetrating into these horizontal seams, however small in quantity, is capable of developing an upward pressure corresponding to a substantial proportion of the static head.

In the dam in question, the possibility of such pressures caused no apprehension, because the thickness of the dam at any plane was fixed largely by the width of the base and the requirement that the down-stream face be shaped to act as a spillway on occasions. When designed to meet these requirements, ample stability was obtained even considering all possible upward pressure, and, therefore, no saving of materials could have been made by water-proofing the horizontal joints. The existence of such leakage is objectionable, however, because it is unsightly, induces doubts as to the quality of workmanship, tends to impair public confidence in the stability of the structure, and results in losses of water, however small, which the dam is supposed to conserve. Various water-proofing fabrics impregnated with bituminous, or asphaltic compounds, or such compounds rolled out as sheets, are available. It is believed that narrow sheets of such substances or of metal can be placed so as to span the joints between pours without the labor and expense incident to the use of the continuous membrane proposed by the author and that such treatment of the joints will prevent visible leakage.

The treatment of the vertical joints is more complicated. In this case, there is no overlying weight tending to close the joint and the width of the joints varies with the temperature of the concrete. Leaks through vertical seams will generally be larger than those through horizontal seams. In the dam previously mentioned, the leaks occurring through the joints between adjacent construction sections reached considerable proportions, and in

several places water streams of considerable size issued freely from the down-stream face of the dam. The inspection tunnel in the lower part of the dam, intersected, of course, by the vertical construction joints, could be kept dry only by the use of much more pumping equipment than had previously been thought necessary. The leakage was of such proportions that elaborate and expensive measures to stop it are in progress.

This dam was built without true expansion joints. Several thicknesses of tar paper were used between adjacent sections at the outer margins of the surfaces in contact, but the main bodies of the sections were in direct contact with each other. After one section had been poured and the side forms removed, the surface against which the adjacent section would be poured was thoroughly cleaned; but as the forms could not be removed while the concrete was green, it was impracticable to roughen the surface by scrubbing in the same way as was done on the horizontal joints. Key boxes were nailed on the inside of the forms, however, thus providing for tongue and groove joints between sections. No membrane of any kind spanned the joints between sections, but the writer now believes that provision for seals of some kind should have been provided. It is believed that in future construction metal or asphalt-treated fabric strips might be folded and tacked to the side forms in such a way that half the width of the strip, say, 8 or 10 in., would be embedded in the concrete as poured and the other half lie flat against the form so that when the form is removed, it might be straightened out and become embedded in the adjoining section when poured. This type of construction would prevent the bulk of the leakage and, at the same time, be free from the objections to the use of a continuous membrane as proposed by the author.

The text and Fig. 1* of the paper indicate a proposal that the vertical expansion joints be filled with asphalt. While expansion joints may be needed in pavements, bridge floors, and other structures of limited thickness, and can be readily provided in such cases, their use in a massive structure, such as a dam or a thick wall, is not necessary and would lead to extraordinary construction difficulties unless the asphalt were applied cold in sheets to the side of one section prior to the pouring of the next section against it. In this case, the asphalt sheets would obtain less intimate contact with the concrete than if it were poured hot into the spaces between the sections. Space for the filling is easily obtained in floors, sidewalks, and streets by withdrawing the form between adjacent blocks before the final set of the concrete; but this method is not applicable to heavy walls or dams because of the difficulty of forming the narrow spaces between the heavy sections. Any template or form left in the concrete for this purpose would be gripped so tightly by the concrete that its removal would be very difficult if not impossible. The use of a plastic material, such as asphalt, in such joints does not always serve to seal the joint so as to prevent leakage. The writer has noted in many joints so treated that in cold weather when the joints open the greatest amount, the filling becomes brittle, breaks away from one of the surfaces of the joint and clings to the other, thus failing to close the opening. This applies to the

* *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1574.

filling as ordinarily used in expansion joints. Better results could probably be obtained by placing strips of metal or water-proof fabric so as to be perpendicular to the plane of the joint, with approximately half the width of the strip embedded in each section. Some of the materials that could be used for this purpose are sufficiently elastic to accommodate themselves to the opening and closing of the joints due to temperature changes, but if non-elastic strips are used they can be readily crimped, somewhat in the manner shown by the author in the enlarged view of the joint at Section *A-A*, Fig. 1, thus permitting all possible variations in the width of the joint without over-straining the strip.

The writer concurs with the author as to the desirability of water-proofing a dam especially at the horizontal and vertical joints, but doubts that such measures can be made so effective as to warrant the omission of uplift from consideration in calculating the dimensions of a dam. Such savings as could be made would be of no use unless the base could be water-proofed so as to decrease substantially the required weight of masonry above the base. The use of continuous membranes as proposed appears unnecessarily complicated and could well be abandoned in favor of narrow strips or sheets placed so as to span the joints.

K. E. Hilgard,* M. Am. Soc. C. E. (by letter).†—The interesting and important topics of the porosity of masonry and concrete dams, together with the hydrostatic pressure in seams, the fissures or more or less open joints under and in such structures, and the consequent uplift created, have long since led to occasionally serious discussions and quite contrasting views, as well as to widely differing precautionary measures.

In the author's brief history of dam design it might be recalled that the consideration of uplift in and under masonry dams was first proposed in the United States by the late James B. Francis, Past-President, Am. Soc. C. E., in 1888.‡ He assumed the full hydrostatic pressure in the bed-joint at the base at the heel, uniformly diminishing to zero at the toe. Shortly afterward (in 1889) and quite independently, Mr. Lieckfeldt, using a different assumption, namely, that the hydrostatic pressure in every bed-joint could only exert an uplift to the extent that it exceeded the pressure from the superimposed weight of the masonry, developed and published a complete method of computation.§ In the same year, Mr. H. Fecht, who had built or was building several masonry dams in the Vosges Mountains (Alsace), quite independently of the other two engineers and their assumptions reached the same conclusion as Mr. Francis, applying it to all bed-joints throughout the dam. According to his theory each layer of masonry confined between two bed-joints is subject only to an uplifting force, equal to the difference between the hydrostatic downward pressure in the upper joint and the upward pressure in the lower joint, both acting on this interjacent volume of masonry.‖

* Cons. Engr., Zurich, Switzerland.

† Received by the Secretary, December 9, 1926.

‡ *Transactions*, Am. Soc. C. E., Vol. XIX (1888), p. 159.

§ *Centralblatt der Bauverwaltung* (Berlin), 1889, pp. 397 and 443.

‖ *Zeitschrift für Bauwesen*, 1889, and "Thalsperrenbau," P. Zeigler, Berlin, 1900, p. 144.

The designs of the two highest gravity masonry (concrete) dams in Switzerland ("Barberine" and "Waeggital"), the former slightly curved and the latter built rectilinear in plan and both completed in 1925, were based (by approaching French regulations) on the assumption of 80% of the full hydrostatic head on the up-stream side diminishing uniformly to zero. By observations the engineers were convinced, that with very careful workmanship and subsequent numerous injections of cement at the base and into the underlying rock, in reality little uplift could take effect. The two dams have maximum heights of 266 and 366 ft., and the crowns are at an altitude of 6 300 and 3 000 ft. above sea level, respectively.

Neither dam was specially coated on the up-stream face, but instead a mix richer in cement was used there. The careful proportioning resulting in a maximum of water-tightness was previously determined by experiments on very large and thick concrete disks, which were tested as to their impermeability by water under high pressure. Most of these tests were made under the supervision of a Special Commission of the "Swiss Association for Economic Conservation of Water". In both structures a high degree of water-tightness has been obtained. No interior drainage proper was provided in either dam.

A number of dams have been built in Germany from designs of Professor Dr. Intze and others to prevent uplift in the interior of the masonry. Professor Intze rather insisted on the possibility of producing water-tight masonry with proper material, good workmanship, and a careful coating of the up-stream face; but at Solingen, in the Urft Valley (Eifel Mountains), and at Marklissa, a curtain of vertical drains of porous tile or perforated glazed pipes of 2 to 3 in. clear width was used, spaced about 8 ft. apart throughout the entire height of the up-stream face and 5 ft. from the surface. These emptied into drainage—or inspection—tunnels and thence into a shaft.*

Experiments in Switzerland carried on by the Special Commission mentioned previously and also by a Special Commission of the "Swiss Engineers' Society", have shown that the correct proportioning of cement, water, sand, and gravel, with careful placing of the mixture, is far more effective in producing water-tight concrete than an admixture of any of the many ingredients widely advertised for that purpose.†

As to the means of creating an absolutely water-tight curtain in the interior of the dam by inserting a totally flat membrane of sheet zinc, lead, or copper, as shown in Fig. 1,‡ it is feared or at least considered possible that temperature and moisture on the up-stream side might cause a separation between the membrane and the concrete. As this proposal has not been actually tested, such a test would seem to be highly desirable. It should be extended to an insertion of a differently shaped membrane consisting of a series

* Handbuch der Ingenieurwissenschaften, 1913, 11 Bd., 2 Abt, "Die Talsperren," by E. Mattern.

† "Gussbeton," Erfahrungen beim Schweiz. Talsperrenbau, S. I. A. Verein, Zurich, 1926. (In pamphlet.)

‡ *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1574.

of corrugations similar to the one shown in Fig. 4* of Mr. Gardiner's discussion. This form is now manufactured commercially of sheet iron, but could just as well be produced of zinc, lead, or copper, with soldered or electrically welded joints.

By applying more mortar than coarse concrete near the membrane all interstices could be securely filled, thus securing an interlocking contact between both sides of the membrane without anchors.

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* *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 113.

EXPERIMENTAL DEFORMATION OF A CYLINDRICAL ARCHED DAM

Discussion*

By William Cain, M. Am. Soc. C. E.

William Cain,† M. Am. Soc. C. E. (by letter).‡—The notable solution of "Arched Dams" given by the author§ is the most complete, for this arched dam, that has yet appeared, in that the shear between supposed horizontal arches was included in the analysis. Where the dam is fixed (encastrée) at the base, the tentative method‖ presented by F. A. Noetzli, M. Am. Soc. C. E., which ignores this shear, affords a good solution. This tentative method also applies to very thick dams if correct formulas for deflection are used.¶

However, when the dam is simply supported at the base the tentative method fails, since the line of deflection of a supposed vertical cantilever makes an unknown angle with the vertical. Mr. Smith's method affords a solution. It applies strictly to a dam in a comparatively level valley, and is, perhaps, only roughly approximate when the dam is situated in a deep V-shaped canyon.

In all theories pertaining to segmental arched dams it is assumed that the normal load on the supposed horizontal arches is uniformly distributed. This may not be correct; but it is doubtless a sufficiently close approximation, as thin circular dams have not, hitherto, failed, as they probably would if the normal load were not approximately uniform.

For the cylindrical tank on a level base, the conditions postulated by the author are fulfilled; since, at the same level, they are the same throughout the full circumference. Hence, it was to be expected that the deflections for a rubber tank filled with mercury should agree fairly well with computed stresses, which was found to be the case for the base either fixed or simply supported. The author has done a real service in making these experiments which tend to verify the theory.

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* This discussion (of the paper by B. A. Smith, M. Am. Soc. C. E., published in October, 1926, *Proceedings* but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

‡ Received by the Secretary, December 20, 1926.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

‖ "Gravity and Arch Action in Curved Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

¶ *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, pp. 206–277, "Stresses in Thick Arches of Dams," by B. F. Jakobson, M. Am. Soc. C. E., with discussion and a solution by the writer by the work method.

The writer was particularly interested in the case of the tank simply supported on a conical base the elements of which, as measured on Fig. 1 (*a*),* make angles of about 14° with the vertical. This value seemed so large that it was checked. By differentiating the value of u* and placing $z = 0$ in the result,

$$\tan \phi = \left.\frac{d\,x}{d\,z}\right)_{z\,=\,0} = \frac{w\,d\,R^2}{E_0\,t}\left(\frac{\mu}{\sqrt{2}} - \frac{1}{d}\right)$$

in which, ϕ = the angle that the deflection curve makes with the vertical. Using the numerical values given in Table 2:†

$$E_0 = 10.2;\; E_1 = 12.2;\; \tan\phi = 0.286;\; \phi = 16°,$$
$$E_0 = E_1 = 13.6;\qquad \tan\phi = 0.227;\; \phi = 12°\,47'.$$

The average is 14° 23′, which shows that the angle used is practically correct.

The wire that binds the rubber cylinder around the conical surface evidently causes shear in the rubber, since each horizontal slice of infinitesimal height slips over one just below.

The case is much more complicated for an arched dam, where the foundation may yield appreciably. For simplicity, consider the foundation rigid for the case of the Wooling Dam simply supported at the base. The solution was affected by the writer by aid of Mr. Smith's formulas and the use of Michell's functions. The values of u, M, and T are given in Table 1‡ and are shown graphically in Figs. 6 and 7§ of the writer's discussion of Mr. Noetzli's paper. The deflection curve for the water load makes the angle at the base with the vertical about equal to 0° 4′ 48″, or, in radians, $\phi = 0.00140$. To compute the modulus in shear (G) for E = 2 000 000 lb. per sq. in., assume Poisson's ratio = $\frac{1}{5}$, or $m = 5$, and use the known formula,

$$\frac{E}{G} = \frac{2\,m + 1}{m}$$

Then,

$$G = \frac{5}{12}\,E = \frac{1}{6}\ (5\,000\,000)\ \text{lb. per sq. in.}$$

The unit shear near the base, corresponding to the displacement, ϕ, is,

$$\phi\,G = 0.0014 \times \frac{5\,000\,000}{6} = 1\,166\ \text{lb. per sq. in.}$$

This value is doubtless incorrect, since practically nothing is known of the value of m for thick beams; besides, the voussoir considered is not in pure shear, being acted on by arch thrusts, by loads due to water and the weight of dam, and by couples giving the moments, M on a supposed vertical cantilever.

Since the shear between the supposed horizontal arches, per unit length of dam, is given by Mr. Smith's Formula (2)‖, $F = -\dfrac{d\,M}{d\,z}$, the best solution would appear to be that based on this formula.

* *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1599.

† *Loc. cit.*, p. 1600.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 312.

§ *Loc. cit.*, p. 313.

‖ *Loc. cit.*, Vol. LXXXIII (1919–20), p. 2040.

The writer believes that the angle, ϕ, at the base is caused by shear and has given the following reason for it*: Bearing in mind that the deflection, u, and the arch thrust, T, are both zero at the base, and since T (compression) increases rapidly above the base, the consequent shortening of a horizontal arch near the base gives a displacement of its crown down stream, the displacement increasing with the height above the base. It is plainly a case of shear; each horizontal arch of infinitesimal height slipping over the one just below it. As this subject is not well understood, the author's views relative to it will be appreciated.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 312.

THE CINCINNATI CITY PLAN IS NOW LAW

Discussion*

BY HARLAND BARTHOLOMEW, M. AM. SOC. C. E.

HARLAND BARTHOLOMEW,† M. AM. SOC. C. E.—The speaker has been greatly interested in Mr. Bettman's discussion.‡ There are a number of reasons or arguments that might be set up on either side of this question of placing full or partial responsibility in the hands of the planning commission. There is the fundamental objection referred to by Mr. Bettman, namely, the placing of authority in the hands of a non-elective body. Perhaps in law that is the controlling fundamental consideration. Against the argument of making the plan commission purely advisory might be cited the fact that if left wholly and exclusively within the hands of elective and appointive officials who are not members of the planning commission, the fundamentals of the plan are ignored, unappreciated, and successive officials are not bound to consider them; and, what is more, those elective and appointive officials who change office, of course, invariably like to express their individuality to make a record, as it were, for their particular administration in the particular office which they may occupy. The speaker does not believe there is any uniform panacea that can be advanced, or any method than can be offered, for the handling of this question.

The Ohio law to which Mr. Bettman refers would not apply in St. Louis, Mo., for instance, because in St. Louis the very form of municipal government invariably makes the entire council, mayor, and full administration of one political faith, and they have acted in unison and as a unit on all matters with only the slightest exception since the adoption of that charter in 1914. Of the votes of Council 99% is always all "yea" or all "nay". There is no such thing as even a minority representation or difference of opinion. From which it may be deduced that the matter of the enforcement of the plan is largely one that has to be studied very carefully in every community. It is not one that can be set up arbitrarily to apply uniformly to all cities.

The speaker would also offer the opinion that the execution of the plan is primarily a co-operative enterprise and not one that must or will lead to checks or differences of opinion. It ought to work smoothly and these halts, discussions, and issues ought not to arise if possible. He will make a

* Discussion of the paper by George B. Ford, Esq., continued from January, 1927, *Proceedings*.

† (Harland Bartholomew and Associates), St. Louis, Mo.

‡ *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 124.

prediction with respect to Cincinnati, which is offered purely for purposes of discussion. Mr. Bettman cited three instances where projects have been held up. Within the course of a few years, if many more such instances arise, the Council will become resentful and enough groups about the city will be antagonized to such an extent that there will be sufficient sentiment to cause the council to repeal its action adopting the plan. Perhaps that might be overcome by a referendum, but even in that case, by the same token, a referendum, sooner or later, might be upset.

This brings the present reasoning on this matter to this: That back of all successful work in government—perhaps "political science" is the term—there must be understanding and co-operation. This, in turn, goes back to public opinion. Despite the fact that a plan commission may get weary of the tremendous effort of keeping public opinion up to the pitch of supporting the city plan, fundamentally that is the proper way to secure the continuous execution of a large proportion of the plan. City planning is no different from any other form of public work. It must be carefully watched and supported by an enlightened citizenship if it is not to fail completely or fall into corruption.

The planning commission has two fundamental jobs. One is to study, and study, and study, indefinitely, always perfecting and developing the plan; and the other is to develop public opinion to the point where that plan will be carried out whole-heartedly and intelligently.

With respect to the first of these, if the planning commission is properly constituted the public officials who have charge of the execution of the plan are going to see to it that the major part of that plan will be intelligently executed. For there is certainly no force in public life and public work greater than that of public opinion, which public officials generally wish to respect.

The speaker would like to offer one or two examples, which he has often cited in discussions with planning commissions and groups in regard to this particular idea. There are two cities which have extensive accomplishments in city planning. They are not alone in extensive accomplishment, but they are used merely for the purpose of illustration—Detroit and St. Louis. Both have done an immense amount of work or at least have secured the execution of work by the city government. The Detroit Commission is named because it is probably clothed with more power than any planning commission in America. The charter of the City of Detroit gives the Planning Commission the widest authority. The Council cannot even initiate an ordinance relating to the opening, widening, or changing of the location of a street, the acquisition of a park or of a piece of property for a public building; it must originate with the Planning Commission. The charter goes further and gives the Commission power to prepare and adopt a zoning ordinance, but it has not attempted to exercise this power, which it realizes is probably illegal.

What has happened? The Council has occasionally resented its deprivation of powers. By constant effort the Plan Commission has succeeded in

keeping a reasonable degree of harmony, but without such effort the situation would have been extremely acute and has been at times with respect to certain issues. Yet Detroit, under it, has been able to accomplish a great deal—things which are not always spectacular on paper, but which have a profound total effect on the city at large.

As a contrast to that situation St. Louis has probably the loosest authority for city planning of any city in the United States. The present ordinance is the original ordinance creating the Planning Commission and was passed in the spring of 1912. All engineers know how limited the understanding and appreciation of city planning was at that time, yet the Commission has continued to function ever since with that particularly loose authority. Through intensive technical work, through incessant publicity, it has succeeded to the extent of securing a wide degree of acceptance and accomplishment.

The City of St. Louis has had ten years of experience, one illustration of which might be of interest to Mr. Bettman. Because of favorable public opinion for adherence to the city plan, the Planning Commission received a great degree of public support on projects that were not to the individual interest of certain political leaders. They began to throttle the work by breaking down the personnel of the Commission through purely political appointments. Fortunately, a change of administration came about in the nick of time. There are more ways than one in which the very best drawn plans may go awry.

The Planning Commission not having funds for extensive educational work, a citizens' committee was created to properly advise the public in the event the Planning Commission itself had been swayed into a political point of view.

One more illustration to show the power of public opinion: It is fundamentally important with respect to planning. Mr. Bettman realizes that despite all this authority of law or the provisions of the statute, if there is not an overwhelming public opinion constantly stirred up, constantly alert, sooner or later the plan will fail.

The illustration is this: Many are familiar with the St. Louis bond issue of \$87 000 000 in 1923, among the items of which was a civic center. The bond issue included \$5 000 000 for the acquisition of all the necessary land amounting to about 45 acres, about 7 acres already having been acquired; \$5 000 000 for an auditorium; \$4 000 000 for a court house; and \$1 000 000 for a soldiers' memorial. These sums were nearly sufficient to complete the plan. There were only two additional buildings needed to complete the entire group.

A complete plan had been drawn. It had been published. Almost everybody voted, assuming that the plan was to be carried out more or less as pictured, but the plans of the Planning Commission were not in complete and final detail. An unnecessary amount of labor and expense would have been incurred without the approval by referendum and the voting of funds. The commercial element of the community opposed the location of the court house in the plaza, and organized to keep it in a particular

section of the business district where it would suit their particular interests. There arose a referendum brought by the commercial interests to place the new court house in that section of the business district not in the civic center.

This rather long story is told because it is unique in American cities and is a refutation of the effectiveness of the Ohio method. With the Retail Merchants' Association, every down-town commercial organization, all the banks, and the political machine organized to keep that court house out of the civic center, a two-thirds vote of the Council would have meant loss of the most important building in the civic center. The citizens' committee composed of citizens from all walks of life, with almost no money to spend, with the support of two newspapers out of three, defeated the referendum and maintained the integrity of the plan by a vote of 2 to 1. Here was an example of the power and the importance of public opinion in the enforcement of the plan. The public would not have supported the plan except for the fact that it knew and understood. There is nothing more powerful than public opinion, and the importance of creating that opinion by the planning commission is in danger of being overlooked if too much reliance is placed on supposedly legal checks or safeguards, because sooner or later ways are always found to amend, change, or revise, unless public opinion blocks it.

THE DESIGN, CONSTRUCTION, AND OPERATION OF A SMALL SEWAGE DISPOSAL PLANT

Discussion*

By L. H. Enslow, Assoc. M. Am. Soc. C. E.

L. H. Enslow,† Assoc. M. Am. Soc. C. E. (by letter).‡—With the exception of one or two details the writer will not discuss plant construction and operation except in so far as chlorination is concerned.

Care Given Small Plants.—One of the most striking statements made by Mr. Hudson,§ namely that "an inspection of this [new] plant was made * * * and it was found to be in a [neglected] condition similar to that of other plants in small towns and for which there seems to be no remedy," states truthfully, but sadly, the facts that engineers are facing. The wording in brackets is explanatory and does not appear in Mr. Hudson's sentence.

Probably better results on the whole might be expected of small installations if the town scavenger were told to build a sewage plant and the consulting engineer later retained to operate the plant instead of the reverse practice as at present. The most perfectly designed and constructed plant must be given proper operating care. Until the consulting-designing engineer includes in the contract a clause which retains him to supervise operation of the new plant for the first year, discredit to the designing or supervising engineer will inevitably result through shortcomings in the matter of care given the plant. The city authorities are too often told in the period of promotion of the new sewage project that once built there would be no more expense to speak of and very little attention would be required. The town officials, having listened intently to early given information about the proposed sewage plant, do not quickly lose the impressions made. The least intelligent man on the city payroll is too frequently assigned to the job of "running the sewer plant". Not many of the city employees want the job and, consequently, after neglect resulting in very foul conditions around the plant, including odors and flies, the job of operating the plant is not sought after and every one is perfectly willing to "let George do it".

To overcome such a state of affairs, to make the job of sewage plant operator more attractive, to obtain more intelligent operators, and to create the desired impression on the public and the city authorities after some months of plant operation, it is almost essential that the designing engineer have

* Discussion of the paper by Franklin Hudson, Jr., Jun. Am. Soc. C. E., continued from January, 1927, *Proceedings*.

† San. Engr., The Chlorine Inst., Inc., New York, N. Y.

‡ Received by the Secretary, December 31, 1926.

§ *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1608.

operating charge of the new plant for the first six months at least. Usually, a plant properly started, and an operator properly instructed and trained during subsequent visits, will prove to be a credit to every one interested, whereas plant neglect in the beginning results in sorrow for all concerned and too frequently the consulting engineer is held to blame, primarily because he can not be present to defend himself and state facts.

In designing new plants and preparing the annual budget to cover financing for amortization, operation, and maintenance, too frequently operation and maintenance have been neglected. The engineer should see to it that a sufficiently liberal fund is provided.

These suggestions may be termed idealistic or impractical, but if engineers and sanitarians will "preach" ideals concerning operation of plants, preferably under supervision of the consulting engineer, more towns will actually try the expedient. The plan should really prove feasible if the profession will work for its application. Leading engineering firms are already demanding such consideration from their clients. In the more progressive larger cities, this plan has been adopted already. It should be extended to the smaller towns where supervision, and at least initial direction, are probably more important than in the case of the larger cities.

The writer does not agree with the following statement made by Mr. Hudson: "* * * about all that was required each week was to clean the screens, check the chlorine dosage, and regulate the Imhoff tank." Experience indicates that even the smallest plants should have attention every 48 hours and preferably once daily. Screenings should be removed, the sloping walls of the flow chamber and slots "squeegeed", and the scum in the vents broken up thoroughly at least three times a week, if the plant is to be kept up to a condition which prevents the collection of scum from gaining such headway that it eventually becomes unmanageable and must be removed at considerable expense and with attendant nuisance.

Naturally enough, where chlorination is being depended upon as a part of the treatment, it becomes more important to make daily visits to the sewage plant for the purpose of checking up the chlorinator operation and properly controlling the chlorination. This feature will be discussed subsequently in detail.

Structural Details.—The following comments and suggestions concerning structural details are open to considerable debate, but are worthy of careful thought by the designing engineer, and plant operators as well. The primary requisite of an Imhoff tank is the maximum obtainable sludge-chamber capacity below the slot with a minimum cost of construction. Secondly, the sloping bottom of the flow chamber should be at such an angle that the solid matters will not lodge thereon but will slide down and through the slot into the digestion chamber below.

With this detail in mind, the Engineering Profession has selected for almost universal usage, the slope of 1.5 vertical on 1 horizontal, producing a sloping bottom floor of the flow chamber which forms an angle of approximately 57° with the horizontal.

A triangular beam below (Fig. 7(*a*)), forms a projection on either side of the slot, preventing solids, gases, and liquid from entering the flow chamber through the slot and causing disturbances. At the same time the use of such a beam provides a slot on both sides, increases the ease with which solids may slide through into the digestion chamber, and theoretically provides also a more uniform distribution of solids in the sludge hoppers.

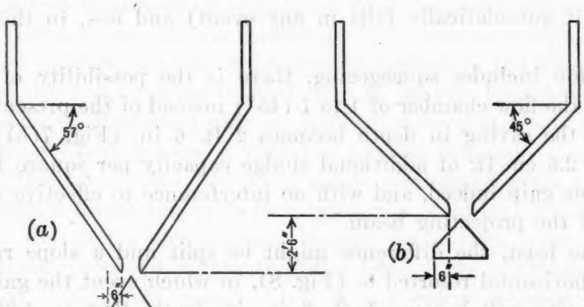

FIG. 7.—GAIN IN SLUDGE CHAMBER CAPACITY BY GIVING LESS SLOPE TO FLOOR OF FLOW CHAMBER.

To reduce in a measure the depth at which a slot would be formed when using the 57° sloping bottom, it has become somewhat common practice to project the apex of the triangular beam up into the flow chamber as is in the plant at Stroud, Okla. The chief reason for such construction is theoretically a good one in that any procedure used to increase the sludge capacity of Imhoff tanks without increased depth of sludge chamber is desirable. Upon analysis, however, in many cases, it will be found that only 5 in., or at best 6 in., additional depth in sludge chamber is secured by projecting the beam into the flow chamber, and this advantage is to some extent offset by the following items.

The slots become more easily choked as a result of this beam face projecting into the flow chamber. Cleaning of the slot and the surface of the sloping walls through effective use of the squeegee is hindered. The squeegee cannot be handled easily or effectively because the projection interferes with its passage through the slot. Another practical objection is that the size of the beam must be materially increased as to the breadth of its lower side in order to effect the proper baffling beyond the edges of the slot. This "over-lap" of the slot, which such beams are designed to produce, should preferably be as much as 6 in., if disturbances due to gas and septic liquor or solids entering the flow chamber are to be prevented. Such undesirable disturbances produce one of the most frequently encountered effects which tend to discredit Imhoff tanks.

It appears from observation of many Imhoff tanks that systematic squeegeeing of the slot and the sloping walls of the flow chamber is essential if satisfactory performance is to be expected and an effluent is to be maintained which has the least possible septic qualities and, consequently, the least oxygen demand and odor.

If, as seems to be true, squeegeeing is an essential part of routine plant operation for satisfactory results, it would appear that greater consideration might be given the idea of constructing flow-chamber bottoms with a reduced slope, with the triangular beam not projected into the flow chamber, and with a greater dependence on the practice of squeegeeing. In other words, tanks should be designed with the understanding that systematic squeegeeing is essential to their satisfactory performance, placing the burden more on operation (where it automatically falls in any event) and less, in this particular, on design.

If operation includes squeegeeing, there is the possibility of utilizing a floor slope of the flow chamber of 1 to 1 (45°) instead of the present 57 degrees. In so doing, the saving in depth becomes 2 ft. 6 in. (Fig. 7(*b*)) instead of 6 in., that is 2.5 cu. ft. of additional sludge capacity per square foot of tank area, a sizeable gain indeed, and with no interference to effective squeegeeing, as a result of the projecting beam.

To say the least, the difference might be split and a slope ratio of 1.25 vertical to 1 horizontal resorted to (Fig. 8), in which event the gain in sludge chamber capacity will become 1 ft. 3 in. in depth, that is, 1.25 cu. ft. of increased digestion chamber capacity for each square foot of tank area. Even this gain is worth consideration.

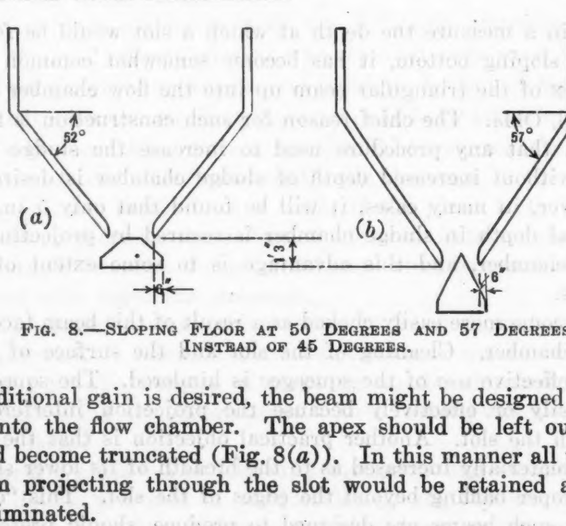

FIG. 8.—SLOPING FLOOR AT 50 DEGREES AND 57 DEGREES, INSTEAD OF 45 DEGREES.

If an additional gain is desired, the beam might be designed as if intended to project into the flow chamber. The apex should be left out, that is, the wedge would become truncated (Fig. 8(*a*)). In this manner all the advantages of the beam projecting through the slot would be retained and the disadvantages eliminated.

At all events, the "over-lap" of slot provided through use of the beam should be as liberal as construction factors permit, preferably 6 in. or more.

Chlorination.—In designing and constructing sewage plants in which chlorination is to become a part of the process it is no longer essential to construct costly chambers to provide long contact periods between the sewage and the chlorine. In cases where some form of contact chamber appears essential, provision for a 15-min. contact, based on the average daily flow, is all that is required. Even this length of contact is not essential where chlorine dosage is regulated efficiently, but the chamber has the advantage of

serving as a "balance wheel" to the operation. In other words, such a chamber takes up minor irregularities of chlorine application; in addition, the variations in the quality of effluent being chlorinated are smoothed out in passage through the contact chamber. The contact chamber, therefore, need not be efficiently baffled except at the inlet where baffling is provided to produce rapid velocity and effective mixing of chlorine and sewage.

In cases where treatment by sprinkling filters precedes chlorination, it has been shown through experiments conducted in co-operation with The Chlorine Institute, that chlorine may be satisfactorily applied to the filter effluent notwithstanding the fact that the effluent may be heavily loaded with suspended humus-like solids. Published data by Keefer* show that solids in the effluent from sprinkling filters possess a very low oxygen demand.

It is interesting to note that such solids discharged from properly functioning biological filters appear to be temporarily stabilized bodies from which the products exerting a spontaneous demand for oxygen have been removed by the oxidation processes within the filter bed. This is assumed in view of the fact that such solids possess little if any chlorine-absorbing properties (that is, chlorine demand).

It has been shown in several experiments since the summer of 1925 that, like the control of water chlorination, efficient sewage chlorination is only accomplished when sufficient chlorine has been applied (regardless of the total quantity needed) to satisfy the chlorine demand of the sewage or effluent and leave a slight quantity of residual or free chlorine in excess to attack the bacteria and spores. This being true, the measure of efficiency of chlorination of sewage effluent may be indicated by the presence or absence of residual chlorine as determined by the orthotolidin test (1 cu. cm. of standard orthotolidin solution to 15 cu. cm. of sewage sample).

The fact that not more chlorine was required to produce residual chlorine in the sprinkling-filter effluent containing the suspended solids than in the secondary tank effluent leads to the assumption and recommendation that chlorine may economically be applied at the inlet of secondary settling tanks. In following this practice the expense of construction of a special chlorination contact chamber can be avoided. The contact is provided and the "balance-wheel" effect secured without the additional cost that is usually considered necessary when chlorination is to become a part of the treatment. Other advantages, in that delayed fermentation of solids is effected and thus the secondary effluent is maintained in a higher state of stability and in that reversion of oxidized compounds in the effluent due to fermentation is retarded and reduced, appear probable when the secondary settling tanks are used also as chlorination or contact chambers.

In construction of secondary settling basins at small plants baffling may be utilized to distribute the solids, but a preferable method is to provide a division wall creating two long narrow chambers rather than a single tank. Such construction allows the complete and effective cleaning of one-half the unit while the other half is in service, and thus by-passing is unnecessary during cleaning operations.

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* *Engineering News-Record*, November 25, 1926.

In secondary tanks mechanical equipment for the continuous or frequent removal of sludge is to be preferred. In the case of the very small plants, however, a separation tank of shallow Imhoff design with floors sloping at 45° requiring weekly squeegeeing of the slopes should prove very satisfactory. The Dortmund type may likewise come in for consideration second to the Imhoff type of separation device, either of which need have but small sludge chamber capacity. The sludge may be transferred frequently to the primary tanks for digestion.

The chlorination of the incoming solids entering such tanks will not prevent satisfactory secondary digestion of these solids. This, likewise, appears to be true in the case of fresh solids entering the primary tanks where pre-chlorination of the crude sewage may be, and frequently is, more economical than chlorination of the tank effluent.

To control chlorination when applying chlorine to the influent of secondary tanks it is not necessary to maintain residual chlorine in the tank effluent. Samples of the chlorinated influent taken at or near the entrance of the tank should show residual chlorine in the supernatant liquor after allowing a 10-min. or somewhat longer period of contact and also the settling out of the heaviest suspended matter.

Chlorine Dosage Required.—No definite figure indicating the chlorine dosage required to provide residual chlorine after a 10-min. contact period can be prescribed for a particular sewage or sewage effluent. Filter effluents are more constant in their chlorine demand than tank effluents or crude sewages. In the case of tank effluents the chlorine demand varies, depending on many factors. Crude sewage is likewise subject to such fluctuations. Of all the factors the effect produced by the suspended solids present constitutes the least.

The greatest of all factors is the degree of septicity and oxygen demand of the sewage chlorinated. In this connection data recently published* by Keefer, of Baltimore, Md., pictures the tremendous increase in the oxygen demand of crude sewage in transit through long outfall sewers in summer and the subsequent decrease in winter.

Regardless of what type or form of sewage or effluent is being chlorinated there is one universal prerequisite if chlorination efficiency is to be had, namely, regardless of the quantity necessary, residual chlorine to a very slight but positive extent (not less than 0.2 parts per million) must be found present after a 10-min. period of contact. The residual chlorine test is simple and is being used by untrained operators successfully. Fig. 9 illustrates admirably the variation in chlorine demand of a tank effluent under study by Mr. W. D. Tiedeman, of the New York State Department of Health, since October, 1925.

It is to be noted that the demand increases in the warm months and decreases markedly in the cold months only to increase again when the warm weather returns. It appears very evident from studies by Tiedeman and others that oxygen demand and chlorine demand are closely allied characteristics and that chlorine substitutes for oxygen to a great measure. The dotted line

* "Bio-Chemical Oxygen Demand of Raw and Treated Sewage," *Engineering News-Record*, November 25, 1926.

indicates the average chlorine dosage for the year which is only 9.60 parts per million. The applied chlorine during the year was distributed, however, so that satisfactory and immediate bacterial efficiency was maintained during the entire year. The dosage was high when necessary, and was reduced when the chlorine demand decreased.

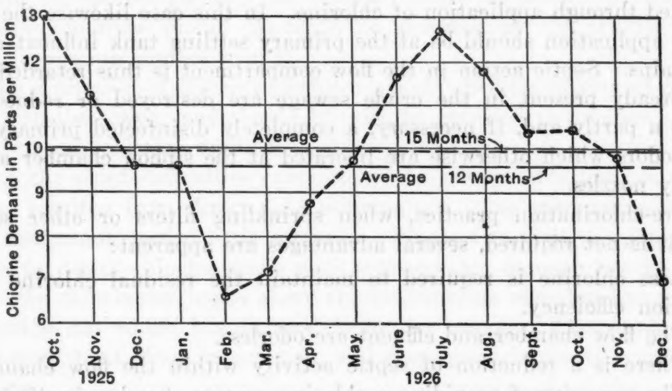

FIG. 9.—SEASONAL VARIATIONS IN CHLORINE DOSAGE REQUIRED AT HUNTINGTON, LONG ISLAND, SEWAGE PLANT.

In the past the practice of sewage chlorination has been that of applying a set dosage continuously, without regard to the variable quality of the sewage, its degree of septicity, and other factors which influence chlorine efficiency and the chlorine demand. Had 10 parts per million been continuously applied (Fig. 9), the efficiency would have been very low practically the entire year except during the colder months, when the chlorine would have been needlessly wasted. On the other hand, had the dosage been required by the health authorities to disinfect satisfactorily under conditions of maximum chlorine demand—that is, 13 parts per million in this case—the sewage would have been over-chlorinated the greater part of the time and the cost would have been excessive.

Further work has indicated that not only does the chlorine dosage required vary during the different seasons, but likewise during each 24 hours. The night flow of sewage being weaker in organic matter requires materially less chlorine to maintain residual chlorine than the day flow. Consequently in sewage plant operation chlorine may be saved by reducing the dosage from month to month when adopting a daily control test. Moreover, added chlorine economy will result if each night the dosage is reduced to provide only the desired residual in the night flow and is increased in the early morning to care for the day flow.

In small plants this is not of importance as far as total cost reduction is concerned, but for disinfection efficiency, residual chlorine tests made daily or at least each 48 hours are essential. It is this feature, in addition to the other matters mentioned, which make it apparent that even small sewage plants require attention from the operator more frequently than once weekly.

Pre-Chlorination of Sewage.—At sewage plants where tank treatment only is given and disinfection is required, it is frequently more economical to apply chlorine at the inlet of the settling tanks. All things being equal, regardless of the content of suspended solids, the fresher the sewage the less the chlorine demand. In plants having secondary treatment such as filters, the odor nuisance is frequently appreciable. Such odors may be reduced and finally eliminated through application of chlorine. In this case likewise the point of chlorine application should be at the primary settling tank influent or at the sewer mains. Septic action in the flow compartment is thus retarded and the odors already present in the crude sewage are destroyed or reduced. The result is a partly and, if necessary, a completely disinfected primary effluent void of odors which otherwise are liberated at the siphon chamber and from the spray nozzles.

In pre-chlorination practice, when sprinkling filters or other secondary treatment is not required, several advantages are apparent:

1.—Less chlorine is required to maintain the residual chlorine and the disinfection efficiency.

2.—The flow chamber and effluent are odorless.

3.—There is a reduction of septic activity within the flow chamber.

4.—The necessity of providing a chlorine contact chamber is eliminated as the tank serves in this capacity.

5.—From present appearances, pre-chlorination does not hinder digestion of the solids dropping through the slot and into the digestion chamber.

6.—Foaming of Imhoff tanks appears to have been controlled at certain plants through pre-chlorination.

7.—Apparently, there is an actual reduction of the oxygen demand of the effluent which can be attributed to chlorine. This is not merely a delayed (retardation of) oxygen demand, but a permanent and actual reduction varying between 25 and 50% of the demand of the unchlorinated effluent.

8.—Flocculent and finely divided solids, such as are frequently discharged in tank effluents, are penetrated and effectively disinfected when residual chlorine is maintained.

Method of Chlorine Application.—Application of chlorine through diffusors, that is, direct gaseous chlorine application, to sewage should be resorted to only when it is not practical to secure a water supply or a supply of clarified sewage for the purpose of operating the solution feed equipment.

Loss of chlorine gas escaping at the surface above the diffusors is objectionable for many reasons. Not least among these is the increased quantity of chlorine required to obtain efficient disinfection, because a good part of the gas is lost in the surrounding atmosphere. Property destruction and nuisance occur in numerous instances, also. These features add to the cost of the chlorine required and materially add to the cost of maintenance and upkeep of equipment. If the dosage of chlorine required is relatively small, as was the case at Stroud, Okla., namely, 3.5 parts per million, or less, a serious loss of chlorine above the diffusors does not occur. When larger dosages are required, such as would be true for tank effluents and crude sewage, the chlorine loss becomes an important item.

If it is essential to apply chlorine directly through diffusors, a relatively great number of them must be provided regardless of the depth of submergence below the liquid surface. In addition, the chamber in which the diffusors are placed should be sealed above to prevent any escape of the undissolved chlorine bubbles which reach the surface. In other words, provision should be made for trapping the escaped gas in a limited space above the sewage surface. Such accumulating chlorine should gradually be absorbed by the rapidly moving current of sewage in contact with it.

Feeding a solution of chlorine produces a more effective mixing of dissolved chlorine with the sewage and should likewise be expected to produce a greater efficacy with less chlorine applied.

One of the chief difficulties encountered in sewage chlorination at small plants has resulted from direct feed chlorination. The assumption has been that the equipment needed little attention. Neglect to visit the plant results in failure to detect chlorine leaks until considerable damage has been done. Likewise, the chlorinator house above the chlorination chamber becomes filled with chlorine gas which has escaped from the sewage. The result may be damaged equipment, final inoperation, and consequent lapses in continuity of disinfection.

Cost of Chlorination.—From what has been previously stated concerning variations in the chlorine demand of sewages or effluents, it becomes apparent that "each tub rests on its own bottom" and the cost of chlorination will depend on the quality of the sewage treated. One very evident fact has been established, however, that is, the better the attention and care given to maintenance of the settling tanks, the less will be the chlorine required. This is because of the more satisfactory quality of the tank effluent produced.

It can be shown that the cost of chlorination may be reduced by applying residual chlorine control. The total saving in the case of small plants is not appreciable. For larger plants it may be considerable. It has been found by Tiedemann that, volume for volume, the night sewage flow requires in some cases less than one-half the quantity required during the day. In view of the decreased volume and coincident decreased chlorine demand a saving of chlorine results if it is reduced at night to provide only the requisite quantity of residual chlorine. In the early morning it should again be increased to insure a residual in the stronger sewage.

From the case illustrated on Fig. 9, it is apparent that without chlorine dosage control no knowledge would have been had to show that a dosage lower than 12 parts per million would prove effective for a part of the year. Consequently, to have satisfied demands that a minimum dosage of chlorine to care for all conditions during the year be maintained, would have resulted in the continuous application of not less than 12 parts per million.

With dosage control through use of the residual chlorine test applied daily at a period of maximum flow, the dosage was varied to suit conditions. The maximum applied was approximately 13 parts, and the minimum only 6.5 parts, per million. The average for the entire year was only 9.60 parts per million, that is, 3.40 parts, or 28.3 lb. per 1 000 000 gal. less than would have

resulted had the constant dosage of 13 parts per million been specified as a safe minimum.

Applying this method of reduction of chlorination cost to a plant treating only 250 000 gal. per day at periods of peak flow, a saving of $(28.3 \times 0.25 \times 365)$, or 2 580 lb. of chlorine per year would result. Moreover, the sewage would be effectively disinfected under all conditions and changes in the rates of flow. Applying the same reasoning to a plant that has a peak day flow of only 5 000 000 gal. per day, the annual saving in chlorine would be 51 600 lb., which, at 5 cents per lb., amounts to a reduction in cost of chlorination of \$2 580 per year.

As previously stated, the night flow of sewage reaching a plant is not only of less volume but has only one-half to two-thirds the chlorine demand of the day sewage during stages of peak flow. Assume a case of applying chlorine from 7:00 A. M. to 7:00 P. M. at a rate to maintain residual chlorine at all times, including the peak flow of 5 000 000 gal. per day. Assume that the flow at 7:00 P. M. has dropped to 3 000 000 per day, with a lowered chlorine demand 66% of that of the peak day flow and that the chlorine is reduced to a rate to produce a residual at that hour and not increased again until 7:00 A. M. the following morning.

During the hours, 7:00 A. M. to 7:00 P. M., the chlorine applied would have been at the rate of (9.60×8.33), or 80 lb. per 1 000 000 gal. for 12 hours. During the night flow the rate would have been only (80×0.67), or 53.5 lb. per 1 000 000 gal.

The total 24-hour consumption, then, instead of being (5×80), or 400 lb., would have been:

$$\text{7:00 A. M. to 7:00 P. M.} \ldots\ldots\ldots \left(80 \times \frac{5}{2}\right) \ldots\ldots\ldots = 200 \text{ lb.}$$

$$\text{7:00 P. M. to 7:00 A. M.} \ldots\ldots \left(53.5 \times \frac{3}{2}\right) \ldots\ldots\ldots = 80 \text{ lb.}$$

$$\text{Total for 24 hours} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 280 \text{ lb.}$$

The saving in chlorine resulting from the 12-hour testing and the change of dosage twice daily would have been 120 lb. per day. With chlorine costing 5 cents per lb., the annual saving effected would have been $(120 \times 365 \times 5$ cents), or \$2 190.

Assume now that the plant treats a flow of 5 000 000 gal. per day (peak flow) with 12 parts per million (500 lb. per day) applied 24 hours per day continuously, during the year, and compare this consumption with that of the same plant placed on residual chlorine control with only two daily tests made at 7.00 A. M. and 7.00 P. M., namely 280 lb. per day. There appears a saving of 218 lb. of chlorine per day. The cost reduction would have amounted to $(220 \times 365 \times 5$ cents), or \$4 015 per year.

These illustrations have been cited merely to show that careful chlorine control may result in a reduction in the cost of chlorination, which, at the larger plants, may be sizeable. Hourly residual chlorine tests with dosage

adjustments up or down barely to maintain residual chlorine in the treated sewage, will doubtlessly prove justifiable at the larger plants.

In the case of the smallest plants, chlorine dosage control maintained by making systematic tests during peak flows is justifiable and essential if efficient disinfection is to result. The saving in chlorine in such cases is unimportant because of the relatively small quantity involved. In medium sized plants, automatic chlorinating equipment should insure a reduction in the chlorine consumed. In small plants the installation of automatic equipment is rarely justified. Relatively, the chlorine saving would be small; further, the automatic equipment would require somewhat better attention and care than the manual control equipment.

In instances where the effluent from sprinkling filters is being chlorinated the fluctuations in chlorine demand during the year are naturally not expected to be as great as for tank effluents or crude sewage. However, for the purpose of obtaining disinfection efficiency with maximum chlorine economy residual chlorine tests should be applied.

At the Stroud, Okla., plant, for example, the chlorine dosage was initially 5 parts per million which, later, was reduced to 3.5 parts per million. The chlorine is applied to the effluent of the secondary settling tank. The reason for the dosage reduction was that nuisance was created as a result of excess free chlorine in the effluent. The fixed average dosage of 3.5 parts per million fails to produce the nuisance, but in the absence of residual chlorine determinations it is impossible to state that such a quantity is not wasteful of chlorine during certain periods and, likewise, that it is not deficient at others.

As is true of chlorination of potable waters, and swimming pools, it is more of sewage, that the only effective control is that which includes systematic testing for the presence of residual chlorine as frequently as the case justifies.

QUANTITIES OF MATERIALS AND COSTS PER SQUARE FOOT OF FLOOR FOR HIGHWAY AND ELECTRIC RAILWAY LONG-SPAN SUSPENSION BRIDGES

Discussion*

By Messrs. T. Kennard Thomson and H. B. Muckleston.

T. Kennard Thomson,† M. Am. Soc. C. E.—It is to be hoped that Mr. Waddell will omit the statement that the data he gives would enable one to estimate the cost of a suspension bridge in a few hours. Only a very experienced engineer could do that and he only after spending weeks in studying the site and local conditions. Such an engineer would probably object to accepting this information second-hand. The statement might encourage many a business man to think that he could make the estimate himself—or hire an inexperienced engineer to do so.

The speaker once met a president of a bridge company who was not an engineer, but had a copy of the Carnegie Steel Company's "Pocket Companion" and had "designed" a number of plate girders by using the bending moment at the center of the span and assuming that half way between the end and the center the moment would be exactly one-half the maximum. From this he reasoned that he could use two angles having the same cross-sectional area as the cover-plate—allowing him to make the length of the cover-plate exactly one-half the span.

Thousands of other cases could be cited (some involving internationally known disasters) to show the danger of putting tools or formulas in the hands of a novice or inexperienced engineer.

As for accepting information about the site, etc., without knowing the personal equation of the investigator—the speaker once received a letter describing the local condition of what he thought was a proposed building, with a request to wire what his charges would be for a report on it. He was on the point of doing so when something made him pause, and then wire that he would make no report without seeing the site first. No answer was received to that, but, later, he heard that the building had been constructed and had collapsed before he was asked to report. In other words, the architect, more than 500 miles away, was trying to get the endorsement of his plans by a distant engineer.

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* This discussion (of the paper by J. A. L. Waddell, M. Am. Soc. C. E., published in November, 1926, *Proceedings*, and presented at the meeting of December 1, 1926), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York, N. Y.

Why should the author give even an approximate formula for the cost of the foundations? Nothing in the cost of bridge work varies more than this. Many other items besides the depth affect it. For instance, the water might be a calm lake or bay, or a raging torrent; or both. While constructing a bridge across the Ohio River, part of the time the depth was 6 ft. with practically no current, and during the same year it was 106 ft.—a violent flood.

The author gives some reasons against the use of eye-bar suspension bridges. Many such bridges have been built and are still standing—but one point which should carry considerable weight when considering long-span suspension bridges is that, while the assumption would be that the pins would be as free to move as an axle, they are, obviously, never so designed. The result is that the pins sooner or later become almost as fixed as a riveted joint, or the pin-holes become slotted. As a proof of the first statement, tests have been made of the stresses in the eye-bars of suspension bridges, with the result that the testing machine indicated compression on the top of the eye-bar and tension on the bottom of the same bar—showing conclusively that the pins had become bound.

Mr. Waddell suggests a concrete casing not in contact with the steel. This is just the reverse of the speaker's experience, for he has found that where good concrete was in direct contact with the steel, no rusting had started; but that where there was a space between the steel and the concrete, the metal was badly pitted.

To allow only 5% for contingencies, even after the most exhaustive studies, seems absolutely inadequate, for many reasons, such as labor conditions, accidents, national finances, or individual and personal equations. For instance, a very able man, experienced in his line, took a Government contract, estimating the exact amount of money he would require, and made his banking arrangements accordingly. Through no fault of his, payments were temporarily withheld and he had to go back to his bankers who then "wiped him out."

In making these few suggestions, the speaker well knows that it would take pages to point out the many valuable features of this admirable paper.

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).†—As a large part of the total cost of any highway bridge is a direct effect of the weight of the floor itself with its pavement, if any, it would appear that there is here a large field open to invention in devising some type of light weight floor that will not sacrifice the many advantages which the reinforced concrete slab possesses. Assuming a certain strength as necessary in any event, the main requirements are immunity against rot and the risk of damage by fire. The light weight aggregate mentioned by the author is not unlike pumice stone in texture and would seem to the writer to be open to the objection of extreme porosity. This would invite the penetration of moisture to the reinforcing steel with the consequent danger of rust and ultimate failure of the slab.

* Cons. Engr., Vancouver, B. C., Canada.

† Received by the Secretary, December 16, 1926.

Timber floors have been very generally and properly condemned in the past for important highway bridges and their use has been discouraged except in cases where the cheapest possible construction has been imperative. It may be, however, that the fault has been in the way in which the material has been used rather than in the material itself. The objection to timber in bridge floors is its tendency to rot and also the possibility, which in many cases is a probability, of damage by fire which may extend to the bridge itself if not caught in time.

In the great majority of cases decay in timber is the result of attack by fungi. There are other causes but they are seldom operative in open-air construction. Decay may start on any surface where the spores can find lodgment, or it may come from infection prior to the time the material is put into place. Some sort of antiseptic treatment is about the only means of guarding against it.

While fire may start on any exposed surface, it can not maintain itself unless the oxygen supply can be kept up. Conditions are favorable to continued combustion on the under side of horizontal slabs or in crevices that extend through the mass. They are unfavorable on the upper side or in blind crevices where air circulation is impossible. The ordinary type of timber floor construction has been such as to invite decay and to encourage combustion. The planks have been laid flat in one or more layers with plenty of air spaces between them so as to offer exceedingly favorable breeding grounds for fungi and the most favorable conditions possible for propagation of fire.

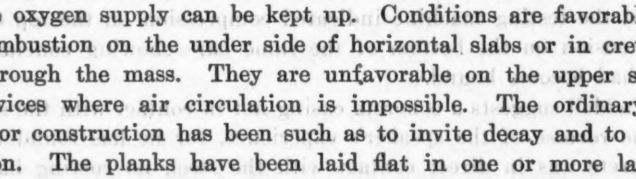

Fig. 22.—Timber Floor Construction.

The type of timber floor construction illustrated in Fig. 22 was suggested to Mr. Waddell some time ago by the writer in connection with a particular bridge problem where cost was an important factor in the decision as to whether or not to build a bridge. The floor planks should be sawn and dressed before seasoning to avoid checks as far as possible. They should be kiln-dried and then saturated with creosote, or other antiseptic, to the limit of their capacity of absorption under the vacuum and pressure process. Before laying, the faces in contact should be blotted dry and painted with a film of asphalt or a similar material, and laid with the paint still soft. A wearing surface of bituminous concrete will serve as a protection from dirt and the ever-present cigarette and cigar butt. A floor of this construction would weigh about 50 lb. per sq. ft. as against 98 lb. for ordinary concrete or 82 lb. for the light weight aggregate.

The main requirement in the anchorage, apart from architectural considerations, is weight and, in many cases, this might be provided by a cheaper material than concrete. The writer offers the suggestion that the anchorages be made in the form of a reinforced concrete box filled with sand, or sand and gravel, which could be pumped into place at small expense. The boxes could be built in much the same way as elevator bins by the use of moving forms.

In respect to the tower encasements which the author has included, the writer fails to see the necessity of ever going to this expense. Their only purpose could be to satisfy the requirements of æsthetics and although there may be a lingering idea that stone towers are the only proper thing for a suspension bridge, the writers believes this is more a result of tradition than of any real improvement in appearance. Properly treated, steel towers can be made quite as effective, possibly more so, than masonry. The contrast between the Brooklyn Bridge and the Manhattan Bridge is not at all to the disadvantage of the latter; and for a horrible example of the effects of disguising the truth there is the Tower Bridge in London. The towers of that bridge were supposed to be designed to harmonize with the massive construction of the ancient Tower of London close by, but it cannot be said that the desire has been realized; at least, not in the opinion of the architects of those parts if one can trust the published remarks of one of them. Mr. Heathcote Statham, a British architect, condemns it as,

"An example of a different kind; it represents the vice of tawdriness and pretentiousness, and of falsification of the actual facts of the structure. * * * All architects would have preferred the plain steel structure to this kind of sham. If iron is designed on good lines it will look better in itself without these gewgaws".

UNIT STRESSES IN STRUCTURAL MATERIALS

A SYMPOSIUM

Discussion*

By Messrs. Thomas K. A. Hendrick and F. N. Menefee.

Thomas K. A. Hendrick,† Assoc. M. Am. Soc. C. E. (by letter).‡—It is suggested that a tabular compilation be prepared giving, for various structures of steel and reinforced concrete, the design unit stress, the actual field data on deformation secured by a strain-gauge, and the computed actual unit stress. A great many structures, being erected every month, should be available for first-hand data of this sort. Perhaps a great many data have already been compiled, but are not available. A thorough compilation covering all sections of the country would demonstrate the desirability of increasing or decreasing the design unit stress. The new strain-gauge designed by the U. S. Bureau of Standards would be of great value for this work.

F. N. Menefee,§ M. Am. Soc. C. E. (by letter).‖—In discussing the subject of unit stresses in structural materials it seems to be fundamentally correct to say that, as the theory of design approaches perfection and knowledge of the forces which come upon structures becomes more complete, it is possible to reduce the factor of safety or increase the allowable unit stresses. In addition, there is, of course, the question of the improvement in the manufacture of structural materials with reference to their strength in resisting the various forces which come upon them and the uniformity and dependability of materials.

During the past few years there seems to have been a revival of the more refined mathematical theory with reference to secondary stresses and the stresses in indeterminate members. While these studies and researches are going on, it seems timely to invite attention to the fact that there are two other phases which at present exercise a considerable control over the stresses in a structure and which are quite often overlooked. These must be carefully considered and studied to bring them up to the same standard of perfection as the theoretical portion of the subject.

The first of these has to do with the modulus of elasticity of the material. As time goes on and American steel mill practice is improved, the material turned out is, of course, more uniform and the modulus of elasticity

* Discussion on the Symposium on Unit Stresses in Structural Materials continued from January, 1927, *Proceedings*.

† Civ. Engr., New Rochelle, N. Y.

‡ Received by the Secretary, December 31, 1926.

§ Prof., Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

‖ Received by the Secretary, January 3, 1927.

more nearly a constant quantity; but the writer ventures to say that at present the modulus of elasticity of structural steel varies between 5 and 10%, and concrete varies as much as 50 per cent. Needless to say, refinement in structural theory will not produce greater accuracy in results than is obtained in such factors as the modulus of elasticity.

The second phase is the state of present practice with reference to structural connections. It is possible, with very careful attention to details of support and loading, to verify theory with reference to deflection and stresses in beams if the beam is long relative to its depth; but very few beams in actual practice get the same ideal or academic attention as the laboratory beam. The connections between beams and columns are of so many different kinds and standards are so silent on this subject, or so often disregarded, that the stresses in a great many structures are very different from what would be obtained theoretically. The writer has measured strains in structural steel (the most uniform building material nowadays) and has found, for instance, that the pouring of a concrete mat produced a fluid condition which in turn produced a lateral flexure of a large, heavy I-beam. This lateral flexure was not contemplated by the designers; it produced a compression in one side of the lower flange, which was supposed to be in tension. Diagonal struts, designed for the purpose of taking compression, at least when the load was static, were, in some cases, found to be in tension, showing the influence of secondary stresses due to unequal settlement of foundations or to temperature variation during the erection of the structure.

Progress is desirable, also an increase in allowable unit stresses; but with the extremes to which structural engineers and architects are going in the matter of high buildings, with almost certain competition in their production, disregarding local foundation conditions in some cases, with the lack of accurate knowledge as to the modulus of elasticity of steel and concrete, and with the disproportionate amount of study that has been spent on structural connections, the writer is forced to take the conservative side of the question with reference to increase in unit stresses until such time as there is a better balance in the study of all the controlling phases of the structural questions.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JOSÉ ANTONIO CANALS, M. Am. Soc. C. E.*

DIED FEBRUARY 28, 1926.

José Antonio Canals was born in Guanabacoa, Cuba, on February 20, 1859. His parents removed to Porto Rico shortly after his birth. He attended the College Seminario of San Juan, Porto Rico, conducted by the Jesuit Fathers, and received his degree of Bachelor of Arts in 1874. Soon afterward he left for Belgium where he attended the School of Civil Engineering of the University of Ghent, graduating therefrom in 1883, and returning to Porto Rico the same year.

From 1883 to 1885 Mr. Canals was in charge of the survey and design of an irrigation system for the District of Guayama, Porto Rico, of which the estimated cost was $500 000 and, in 1886, he designed improvements and the extension of the water-works at Guayama.

From 1886 to 1888, still under the Spanish Government, he served as City Architect of San Juan, leaving this position to accept one with "La Société d'Entreprises et de Constructions des Colonies Espagnoles", General Contractors of the Porto Rican Railroad Company for the construction of 194 km. of road, with which he remained until 1895, having been engaged as follows: From January, 1889, to 1890, he served as Assistant Engineer; from 1890 to 1892, as First Assistant to the Chief Designing Engineer; from 1892 to 1893, as Chief Designing Engineer; and from 1893 to 1895, as Representative of the Company in connection with the liquidation of its contract.

Mr. Canals then accepted a position with the Porto Rican Railroad Company, acting as Chief Engineer of the rolling stock, from 1896 to 1898, at which time he was transferred to the Maintenance of Way Department. During 1898, after the Spanish-American War, the Americans took possession of the Island of Porto Rico and Major General Brooks, knowing the interest and skill of Mr. Canals, appointed him Engineer of Public Works of Porto Rico, in charge of surveys and the construction of roads. He held this position until 1900, at which time he returned to the Porto Rican Railroad Company as Chief Engineer on the repair and reconstruction of bridges.

In 1902, Mr. Canals was appointed City Engineer of San Juan, by Governor Hunt, and as such superintended the repair of the water-works and studied conditions to prevent the waste of the water supply. Under great difficulty and in the face of much bitter opposition he succeeded in introducing water meters for the conservation of the water supply and thus curtailed the enormous expense which previously drained the Municipal Treasury. This

* Memoir prepared by J. M. Canals, Esq., San Juan, Porto Rico.

measure was passed only after much heated discussion, attacks by newspapers, and denouncement by the public, but his integrity of character never wavered and he installed the meters, the consumption decreasing from 235 liters per capita per day in 1902 to 80 liters in 1905. At the same time he was engaged in completing the sewerage system and street paving.

From 1905 to 1919, Mr. Canals had an office as Consulting, Contracting, and Designing Engineer, and was in charge of the construction of a number of large buildings. He also saved the Loiza Sugar Company and the Canóvanas Sugar Company about $1 500 000 by preventing the replacement of their 75-cm. railway by a 1-m. railway. This was an important piece of work, for a bill was before the House of Representatives demanding the removal of the 75-cm. tracks.

In 1919, his services were again required by the municipality of San Juan on account of an insufficient water supply due to an extremely dry season. Mr. Canals' strength of character against great opposition and his unwavering purpose in giving his best to the people in 1902 were remembered and fully appreciated and during this crisis he was called to advise and accepted the position of Superintendent of Public Works.

In 1920, he left the Municipal Service and re-opened his office as Consulting Engineer, but, in 1923, he was recalled again by the City Government to consult with other prominent engineers on the revision of the project of new water-works for San Juan. Subsequently, he was appointed Chief Engineer in the Office of City Improvements, taking charge at the end of 1923 of the Office of Director of Public Works.

In March, 1924, the party that was in power had to yield to the opposition. The latter, keenly aware of Mr. Canals' sterling worth, felt that it could not do without his assistance and offered him the position of Consulting Engineer of the City of San Juan, which position he held until his death. During this last period of his life he was engaged zealously in the location, construction and re-adjustment of the new water-works of San Juan.

In addition to his various positions in the engineering field, Mr. Canals occupied a number of important offices in the Insular and Municipal Government. On April 14, 1904, he was appointed by Acting Governor Charles Hartzell of Porto Rico, a member of the Superior Board of Health and on September 21, he was appointed a member of the Municipal Council by Governor Arthur Yager. He was also made a member of the Workmen's Relief Commission on May 27, 1918. On December 18, 1922, he was appointed by Acting Governor J. B. Huyke as President of the Mediation and Conciliation Commission.

Outside his official duties, Mr. Canals' interest was especially in his home where he died on February 28, 1926. He is survived by his widow, a son, and three daughters.

In 1911, he was President of the Porto Rican Society of Civil Engineers, having been at the time of his death an Honorary Member of that Society.

Mr. Canals was elected a Member of the American Society of Civil Engineers on May 3, 1905.

GEORGE STOVALL EDMONDSTONE, M. Am. Soc. C. E.*

Died March 23, 1926.

In the passing of George Stovall Edmondstone on March 23, 1926, the Engineering Profession lost a member of great ability, and the community, a gentleman of the old school. To him honor was of first importance and loyalty to principles and friends a predominant quality. To many younger engineers he was an inspiring influence in fixing high ideals of professional and personal integrity.

George Stovall Edmondstone, always known as "The Captain" because of his association with Army work as Civilian Engineer, was born in Augusta, Ga., on January 22, 1864. The Edmondstone family is of Scotch descent; the first member to become an American, settled in Charleston, S. C., in the Eighteenth Century. George Stovall Edmondstone's father, Henry Edmondstone, was a cotton broker in Augusta, whose business was ruined by Sherman's famous march to the sea. He afterward for many years was Cashier of the Georgia Railway.

George Stovall Edmondstone's professional education was obtained in part at the Carolina Military Institute at Charlotte, N. C., from which he was graduated in 1882. After a year spent in railroad construction work in his native State he continued his engineering training under the late James B. Francis, Past-President, Am. Soc. C. E. Following his hydraulic work under Mr. Francis, he served as Assistant City Engineer of Augusta for several months, being engaged in sewer and hydraulic work. The years from 1886 to 1890 were spent as Construction and Designing Engineer for a New York firm financing and building water supply systems for various cities, including Greencastle, Md., and Defiance, Ohio. He served as City Engineer of Defiance from 1890 to 1892.

Leaving municipal work in 1892 he went to Wyoming where for five years he was in private practice, engaged chiefly in irrigation and water power projects. In 1900 he returned to municipal work and for four years held the position of Bridge Engineer for the City of St. Paul, Minn. The bridges designed and constructed by Mr. Edmondstone included those on Western Avenue, Arcade Street, Rice Street, and Grand Avenue. From 1904 to 1907, he was Construction Engineer for the United States Government at Forts Keough and William Henry Harrison, in Montana. This again was chiefly hydraulic work.

Three years, 1907 to 1910, were spent in private practice with headquarters at Helena, Mont. His work at this time was varied, but was chiefly hydro-electric and structural. During this period he was Construction Engineer for the Missouri River Power Company, reporting on the failure of the Houser Lake Dam. Water-right determinations for irrigation companies and irrigation construction occupied part of his time during this period.

In 1911 Mr. Edmondstone undertook the investigation, plans, and estimates of the Clear Lake (McKenzie River), Oregon, water power project of

* Memoir prepared by Philip H. Dater, M. Am. Soc. C. E.

the Oregon Electric Company. Three years were spent on this work. The project is an outstanding testimonial of his professional ability. He transformed a dubious project into one of certainty, materially increasing the power capacities, and greatly reducing the estimated cost. This accomplishment is typical of much of his other work.

From 1914 to 1917, he was in charge of the design and construction of important railroad grade separations in Portland, Ore., notably the Sullivan's Gulch grade separation of the Oregon-Washington Railroad and Navigation Company. The following two years were spent as Construction Engineer in charge of building the Portland Municipal Grain Elevator and Terminal No. 4 at St. Johns, Ore. From January, 1921, to July, 1922, he served as Engineer for the Portland School Board, being in charge of school properties.

In 1922, Mr. Edmondstone engaged in private engineering practice and in contract work in which he continued until his death. The fuel conveyor plant of the Long-Bell Lumber Company, at Longview, Wash., is one of the accomplishments of this period of a busy and useful life.

In 1899 he was married to Emilie von Bergen, at Iowa City, Iowa. Mrs. Edmondstone, a son, George Stovall, and three daughters, Claire, Annie, and Elizabeth, survive him.

No outline of Mr. Edmondstone's life would be complete without mention of his active interest in the Masonic fraternity. Few members of the order were better versed in its history and traditions, nor more loyal in carrying its teachings into practice.

Mr. Edmondstone was elected a Member of the American Society of Civil Engineers on November 27, 1917.

CHARLES FROMMER, M. Am. Soc. C. E.*

Died October 13, 1923

Charles Frommer, the son of Jacob and Mary Dorothea Frommer, was born in Belvidere, N. J., on December 5, 1873. Both his father and mother came from Germany and lived in Belvidere for many years. After his father's death, his mother moved to Philadelphia, Pa., where she provided a home for her family.

Mr. Frommer received his early education in the public schools of Belvidere, leaving his home town in 1888 to make his way in the world. In the same year he entered the employ of the Lehigh Valley Railroad Company, as a Rodman, and advanced, successively, through the positions of Chainman, Transitman, and Draftsman.

In 1892 he left the Lehigh Valley Railroad Company to accept a position with the Pencoyd Iron Company as a Structural Draftsman on the design of buildings and bridges. Mr. Frommer was with this Company a little more than two years until in September, 1894, he entered the employ of the City of Philadelphia as Draftsman in the Bureau of Surveys, where he advanced through the various grades.

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* Memoir prepared by H. T. Shelley, M. Am. Soc. C. E.

In the fall of 1897 he enrolled in the night school of Drexel Institute, and was graduated in 1899, having taken the course in Building Design and Construction.

Mr. Frommer continued as a Draftsman in the Bureau of Surveys of Philadelphia until October, 1904, when he was advanced to the position of Assistant Engineer in Charge of Sewer Design. He continued in this work until January, 1916, when he was advanced to be Principal Assistant Engineer of the Bureau of Surveys. In this position he had direct contact with, and supervision of, the engineers who were in charge of the various divisions of the Bureau of Surveys. He continued as Principal Assistant Engineer until his death on October 13, 1923.

During the latter part of Mr. Frommer's service with the Bureau of Surveys, there was an interim of a year and one-half when the Bureau was without a Chief Engineer, and he was designated Acting Chief Engineer during that period.

Mr. Frommer was an ardent and indefatigable student and soon developed the qualities of a successful engineer. He was of a genial and generous disposition and was beloved and respected by those who came in contact with him.

On June 28, 1922, he was married to Katherine Muller, of New York, N. Y., who, with one child, a son, survives him.

Mr. Frommer was elected a Member of the American Society of Civil Engineers on June 4, 1913.

VLADIMIR VASILLIEVICH GORIACHKOVSKY, M. Am. Soc. C. E.*

Died August 9, 1926.

Vladimir Vasillievich Goriachkovsky was born at Kiev, Russia, on July 12, 1869. After graduation from the University of Kiev, he entered the Technical Institute in Petrograd, Russia, of the Ministry of Ways and Communications, from which he was graduated with the degree of Civil Engineer in 1897.

Mr. Goriachkovsky's first work in engineering was as Resident Engineer on the construction of the Sura River Bridge for the Moscow-Kazan Railway and, later, as Engineer in the Technical Department.

In January, 1903, he was appointed Resident Engineer by the Moscow-Windau Rybinsk Railway, in charge of grain elevator construction and harbor improvements. Later, he became Superintendent of Maintenance of Way, and Assistant to the General Manager. He remained in the capacity of Advisory Engineer to this railway until control of the road was lost by its owners.

From 1903 to 1910 Mr. Goriachkovsky made many trips abroad to study modern methods in engineering work, including the United States and Canada; Switzerland, during the driving of the Simplon Tunnel; Antwerp, Belgium, during the enlargement of the port; Roumania, during the construction

* Memoir prepared by D. L. Hough, M. Am. Soc. C. E.

of the grain elevator program; and England and Germany, to familiarize himself with cold storage construction and operation.

In 1910 Mr. Goriachkovsky was appointed Chief Engineer of the Altai Railway which was to open that most fertile valley in Siberia—a valley also rich in precious and semi-precious minerals.

As Engineer for the International Bank of Commerce of Petrograd, he built and operated a cold storage plant in that city and was appointed as the Russian Associate Engineer to co-operate with D. L. Hough, M. Am. Soc. C. E., in the development of the Moscow Central Stations Project which the bank and its group had undertaken to finance. In addition to his other activities, he was Associate Engineer and Director of the Arozdayou, Caucasus, Cotton Growers Company and had charge of the construction of its comprehensive irrigation system. He was also Controlling Consulting Engineer to the Russian State Bank for its Grain Elevator Department.

The climax of his career in his native country came during the World War when it became necessary to reach by rail an ice-free port on the Murman Coast of Lapland, the final goal of the Gulf Stream. The necessity for this road was perhaps the most serious engineering matter to be solved at that time by the Russian Government and Mr. Goriachkovsky was chosen as Chief Engineer and General Manager to build 1 000 km. of railroad suitable for heavy traffic through an uncharted territory where vast swamps were encountered and during two Arctic winters. He accomplished the task in eighteen months. For this achievement he received the highest recognition of his own Government, and was decorated with the Cross of St. Michael and St. George by the British Government and made an Officer of the Legion of Honor of France. He had as many as 80 000 workmen employed on this undertaking to whom it was necessary to transport food, equipment, and tools, which in itself was an Herculean task.

Of this achievement the *London Times* wrote on October 30, 1915:

"When on the shores of the Arctic Ocean the first locomotive whistles, Russia will have the right to say that she has once more accomplished a labour of Titans and will be able fully to value the iron will, solid knowledge, and indefatigable energy of the Chief Constructor of the Murman Railway, Engineer V. Goriachkovsky, and his co-workers, great and small."

Mr. Goriachkovsky was not a specialist in one branch of engineering, but in all branches, and had been soundly instructed, as his accomplishments give proof. He was considered to be one of the first, if not the first, engineer of Russia. To these technical qualities were added geniality, tact, and every social gift. He was an entertaining speaker, an interesting companion, and a good friend.

Leaving Russia as late as May, 1918, he at once volunteered his services to the United States War Department, in Washington, and served in the Intelligence Department until the end of the World War. Subsequently, he became associated with the Youroveta Home and Foreign Trade Company for which he designed a chain of packing houses and cold storage plants it was hoped to establish in Siberia.

On March 5, 1919, he presented an interesting paper before the Society entitled "Engineering Construction in Russia and General Conditions in That Country" illustrating his remarks with lantern slides.

In 1920 Mr. Goriachkovsky started on a comprehensive trip in behalf of certain countrymen who wanted his advice as to a suitable location or locations for the colonization of Russian refugees. He visited the Southwestern United States, Mexico, the West Coast of South America, and crossed to Argentina, but in Buenos Aires his studies were cut short by a nervous breakdown. Crossing directly to Europe, he entered a sanitarium in Germany where he apparently recovered from his illness. He then made his home near Lausanne, Switzerland, where he died on August 9, 1926. He was buried in Genoa, Italy. His widow, Xenia Michaelovna, survives him, as well as a large family of brothers and sisters.

Mr. Goriachkovsky was elected a Member of the American Society of Civil Engineers on November 25, 1919.

ASHBEL EDWARD OLMSTED, M. Am. Soc. C. E.*

DIED AUGUST 15, 1926.

Ashbel Edward Olmsted, the son of Ashbel and Emeline (Stanley) Olmsted, was born in East Hartford, Conn., on December 21, 1845. He was descended from James Olmsted, one of the first settlers of Hartford, Conn., who came through the "wilderness" with Parson Thomas Hooker from Cambridge, Mass., in 1636.

At the age of thirteen Mr. Olmsted entered the Hartford High School where he was a student for two years, leaving before his graduation on account of weak eyes. He then assisted his elder brother on the farm, but continued the study of mathematics at home. After an extended visit with relatives in Iowa he returned with the settled idea of becoming a civil engineer. In his twenty-first year he expected to enter Yale College, but on account of a dangerous gunshot wound received while hunting in Iowa, he was obliged to give up his college education. Nothing daunted, however, he continued his studies at home with the help of Hiram Loomis, an engineer of Hartford, and in 1868 secured his first position on a division of the Troy and Greenfield Railroad east of the Hoosac Tunnel, under the late John Waldo Ellis, M. Am. Soc. C. E., Division Engineer.

In 1869, when the Connecticut Valley Railroad surveys were started, Mr. Olmsted held for a short time the position of Flagman, but was soon advanced to that of Rodman, then, as the Transitman failed to appear when the engineering party was made up, he took levels and did transit work until, in 1870, he was placed in charge of a Division from Deep River, Conn., to Long Island Sound, which position he held for three years.

From 1873 to 1879 he did various kinds of engineering work in different parts of Connecticut and in his home town, but again turned to railroad

* Memoir prepared by William J. Boucher, M. Am. Soc. C. E., from data furnished by relatives.

construction in the summer of 1879 when he became Assistant Engineer on the New York and New England Railroad under L. B. Bidwell, Chief Engineer. A year later Mr. Olmsted was placed in charge of the new line in Putnam County, New York, with headquarters at Towners, and, later, had charge of the terminal work at Fishkill-on-the-Hudson.

His next position was on the New York, West Shore, and Buffalo Railroad, where he had charge of twenty-one miles under Mr. Thomas Spencer. In January, 1883, the late Elmer L. Corthell, Past-President, Am. Soc. C. E., advised him to go to Syracuse, N. Y., to interview the members of the firm of Ryan and McDonald (John B. McDonald), which was starting extensive work on the West Shore Railroad. An agreement was made between them, and for a few weeks Mr. Olmsted was with Mr. McDonald at Syracuse, later going to Lyons, N. Y., where thirty-four miles of railroad was being built under Mr. Ryan.

In the fall of 1883, the West Shore Railroad failed, but Mr. Olmsted remained to settle the firm's affairs until the spring of 1884, when Mr. McDonald assigned him to work in connection with the extension of the Baltimore and Ohio Railroad from Baltimore, Md., to Philadelphia, Pa. He was located at Chester, Pa., for two years. While engaged on this work he first met Colonel (later, General) Henry T. Douglas, Chief Engineer of the Baltimore and Ohio Railroad. Thus began an acquaintance and friendship that lasted until the death of General Douglas in July, 1926.

One day in the fall of 1885, when Mr. McDonald asked Mr. Olmsted how soon he could go to Chicago, Ill., he answered "to-night", and by midnight he was on his way to look over eighty miles of heavy work on the Illinois Central Railroad from Elgin to Freeport, Ill. Soon afterward, the firm secured the work and Mr. Olmsted went to Rockford, Ill., where he remained for a year. Later, the firm built fifty-seven additional miles from Freeport, Ill., to Dodgeville, Wis. It was a rush job, the contractor having agreed to build it in four months.

In the spring of 1888, Mr. Olmsted went to Seattle, Wash., a very little known city at that time, to look over work on the Seattle and Eastern Railroad in the Cascade Mountains. In August of the same year, he and his family moved to the Pacific Coast where they lived at one of the largest hop ranches in the country at Snoqualmie, Wash., where he made his headquarters. He was there only a short time as the firm found financial conditions such as not to warrant continuing the construction of the line. He went to Tacoma, Wash., for a short time, but as conditions in the Pacific Northwest were not favorable to a civil engineer seeking employment, he returned East.

During the summer of 1889 he was employed in the coal fields of West Virginia and, later, was with the Philadelphia and Reading Railroad Company, in charge of the masonry work for a large bridge over the Schuylkill River, with headquarters in Norristown, Pa. His last work with the firm of Ryan and McDonald began in 1890 and lasted two years, when he was engaged on the construction of the Western Maryland Railroad, and resided at Williamsport (near Hagerstown), Md.

In 1892 Mr. Olmsted went to Panama in connection with a manganese mine, but remained there only a few months as the climate was too great a strain on his health. In 1893, Mr. McLeod, President of the New York and New England Railroad Company, was trying to get an entrance for that railroad into New York, N. Y., and proposed building about fifty miles of road from Brewster, N. Y., to New York City. It was to be known as the New York, New England, and Northern Railroad. Mr. Olmsted was appointed Chief Engineer, with headquarters in White Plains, N. Y., and had two or three engineering parties in the field, but in December of that year the railroad went into the hands of a receiver and the work came to an end.

Mr. Olmsted's next work was at the Croton Dam, where he endeavored to construct a railroad line from the Town of Croton to the dam. This, however, was not possible as there was no way to force settlements of land damages, but he was able to build a short road from a stone quarry to the work. In the summer of 1894 he located a railroad to a phospate mine in Tennessee.

The years between 1895 and 1899 were spent by Mr. Olmsted as Consulting Engineer for the Central New England Railroad Company and at the Jerome Park Reservoir, which Mr. McDonald was building for the City of New York. A part of the work was the construction of a railroad line from Jerome Park to near Hunts Point, Borough of The Bronx, which was used for transporting the excavated material and disposing of it on the large area of marsh land in that vicinity.

In the fall of 1899, the first subway was offered for bidding by the Rapid Transit Commission of New York City. Mr. McDonald planned to bid on the work and to Mr. Olmsted he entrusted the investigations and studies of this stupendous project. It was indeed a new subject, for at that time only meager knowledge existed of the subsurface conditions of New York's streets. It is true that borings had been made to explore for earth or rock conditions, but the real problem was the location of water and gas pipes, sewers, and electric conduits, the exact data for which were lacking with City departments and utility companies, the drawings accompanying the contract denying responsibility for the accuracy of the information given. Mr. Olmsted spent a month on the study and at its conclusion Mr. McDonald put in the now famous bid for the entire work, the construction of the subway in Manhattan, viaducts in The Bronx, twenty-one miles in all, including the Harlem River Tunnel and Ship Canal Bridge, and the reconstruction, relocation, or restoration of all subsurface structures encountered, for the lump sum of \$35 000 000. There was only one other bid, which was higher.

No contract had ever been offered or awarded for such a large amount of money and certain it is, that none had ever been considered which involved such a variety of work to be done under such uncertain conditions. No praise can be too great for the painstaking effort, for the skill exhibited, and for the keen insight which was necessary in order to grasp the problem and form an opinion of the costs and methods of accomplishing the great task. This Mr. Olmsted did.

After the award of the contract, he was offered the position of Chief Engineer, but felt he should not accept it on account of his health. He was, therefore, appointed Division Engineer, a position which he filled with the Rapid Transit Subway Construction Company (the company organized to take over the McDonald contract) and with the Interborough Rapid Transit Company, until his retirement in 1918. In this position he was actually in charge, under the late S. L. F. Deyo, M. Am. Soc. C. E., Chief Engineer, of the construction work, visiting the various sections, at the beginning, almost every day and, at the office, checking quantities and going over the estimates prepared by the Resident Engineers. Later, Contract No. 2, for the extension of the subway from Ann Street, Manhattan, under the East River to Atlantic Avenue, Brooklyn, was also constructed by the same Company, and Mr. Olmsted performed similar duties as on Contract No. 1. The responsibility for the accuracy of the estimates, and consequently for the financial side of the work, was his. His name appears on the memorial tablets in both the City Hall Station, Manhattan, and Borough Hall Station, Brooklyn.

After his retirement in 1918, Mr. Olmsted returned to his boyhood home at East Hartford, where he read, studied, and worked in his garden until six months before his death, which occurred on August 15, 1926, after a lingering illness. He was married to Emma Leland Strickland, of Essex, Conn., on August 5, 1872, who died in 1903. Mr. Olmsted is survived by an adopted daughter, Ella S. Olmsted.

Mr. Olmsted was of a most agreeable, courteous, and kindly disposition. Those in trouble who applied to him, went away either with substantial assistance or good advice. Painstaking, methodical, careful, and accurate in his work, he was a loyal assistant to all his employers, and at the same time absolutely honest in seeing both sides of the proposition and coming to a decision only after weighing all the evidence impartially.

He was a life-long member of the Congregational Church, a member of the New York Academy of Sciences, and also a member of the Republican Club of Hartford.

Mr. Olmsted was elected a Member of the American Society of Civil Engineers on May 6, 1903.

CHARLES BRADLEY ROWLAND, M. Am. Soc. C. E.*

Died November 1, 1926.

Charles Bradley Rowland was the son of the late Thomas Fitch Rowland, Hon. M. Am. Soc. C. E., and Mary Eliza Bradley Rowland, and was born in the Greenpoint Section of Brooklyn, N. Y., on May 22, 1863. He received his early education in private schools in Brooklyn and New York, N. Y., and later entered the School of Mines, Columbia University, from which he was graduated in 1884, with the degree of Civil Engineer.

* Memoir prepared by Thomas Fitch Rowland, Jr., M. Am. Soc. C. E.

Immediately thereafter he entered The Continental Iron Works which had been carried on by his father for many years, and began his career in the machine shop. This was followed by several years' experience in the drafting room, and, later, he became Vice President of the Company, which position he occupied at the time of his death. As such, he exercised a general supervision over the Company's activities, having been particularly connected with the building of illuminating gas producing plants, supervising the construction of refrigerating machinery, and also special machinery used in the production of paper pulp.

Mr. Rowland was a member of the American Society of Mechanical Engineers, the Society of Naval Architects and Marine Engineers, and was also a member of the Union League Club of New York and the Franklin Institute of Philadelphia.

He was married in 1903 to Alice Cary of Detroit, Mich., who survives him with a son, Thomas Fitch Rowland, 2d, and a daughter, Mary Esther Rowland. He died at his home in Greenwich, Conn., on November 1, 1926, after a brief illness.

Mr. Rowland was elected a Member of the American Society of Civil Engineers on May 1, 1895.

HENRY BEECHER WOOD, M. Am. Soc. C. E.*

DIED SEPTEMBER 28, 1926.

Henry Beecher Wood, the son of Charles Thomas Wood, of Bradford, Mass., and Sarah Jane (Harnden) Wood, of Portland, Me., was born in Woburn, Mass., on August 28, 1854. His ancestors, like those of his wife (the Eatons), were direct descendants of some of the first settlers from England, dating back to early Colonial times. His father was a Civil War Veteran and his grandfather a Veteran of the War of 1812.

Mr. Wood was educated in the public schools of Woburn. He was graduated from the Woburn High School and later entered the Massachusetts Institute of Technology, graduating in 1876 in the course of Civil Engineering. Immediately after his graduation he taught school in Woburn.

In 1880 he entered the employ of the Mississippi River Commission, United States War Department, on topographical work on the river below Memphis, Tenn. He later became Assistant Engineer on Hydrography and Topography, and from 1882 to 1885 was principally in charge of hydrography.

In 1885, Mr. Wood accepted a position with the City of Boston, Mass., on the Boston Water-Works, and was assigned to field work and computations on the construction of the Farm Pond Conduit. During the winter months he was engaged in the development of the Sudbury River water-shed, and on the survey of the Shawsheen water-shed.

From 1887 to 1891, he served as District Engineer in the Sewer Department of Boston, in charge of the design and construction of all sewers for

* Memoir prepared by John R. Freeman, Past-President, Am. Soc. C. E., and Charles T. Main, M. Am. Soc. C. E.

about one-third of the city. He was also in charge of triangulation and topographical development of suburban territory.

In 1891, Mr. Wood was made Secretary and Executive Engineer of the Consolidated Street Department of Boston, which included what was formerly known as the Paving Department, Sewer Department, Bridge Department, Sanitary and Street Cleaning Department, having charge also of improved sewerage, and smoke prevention. He held this position until 1897, when he was appointed Chief Engineer of the Massachusetts Topographical Survey Commission. He was in full charge of town, city, and State boundary surveys under this Commission and became an expert in triangulation work. His duties also included the preparation of an atlas of all boundaries, perambulation of lines, special river and pond surveys, etc.

On June 6, 1901, this work was transferred to the Board of Harbor and Land Commissioners, and under it Mr. Wood continued to be in full charge of his former duties until their completion on May 10, 1916, the final atlas of boundaries having been completed on February 11, 1916.

He also had full charge, in 1902, of the survey of the inland canal, known as the Brockton Canal, which extended from Weymouth, Fore River, through Brockton and Taunton to Fall River, Mass., as well as the Mount Tom Reservation Survey, Nashua River Survey, and other special work of that nature.

From July 20 to October 2, 1915, Mr. Wood's services were temporarily loaned to the Massachusetts Highway Commission and from May 10 to September 11, 1916, to the Bureau of Statistics to measure and complete the water areas in the State for census purposes.

Under the Division of Waterways and Public Lands, created on August 3, 1916, he continued as Assistant on general and research work. In 1918 he was engaged on the compilation and publication of a "Report on Water Resources of Massachusetts". On January 23, 1920, he was assigned to the care and custody of plans and records of the Department of Public Works, indexing, classifying, and regulating them in the Document Room at Commonwealth Pier No. 5.

After his retirement from the State service on August 29, 1924, Mr. Wood was engaged as Consulting Engineer on minor work for the Town of Belmont, Mass.

He presented a good summary of his principal State work in connection with the boundary surveys in a paper* read before the Boston Society of Civil Engineers on April 25, 1898, and published by the Society. In connection with the survey work it is interesting to note that Mr. Wood was continually experimenting with the design of stadia rods, both as to the most practical hieroglyphics and as to color. He was highly skilled in all phases of triangulation surveys and it is said by other engineers that no one outside the United States Coast and Geodetic Survey was more competent or skillful than Mr. Wood in this work.

In 1891 he became a member of Mount Horeb Lodge, A. F. and A. M., and, in 1912, of the Belmont Lodge at Belmont. He was also a member of the

* "State, City, and Town Boundaries," *Proceedings*, Assoc. of Eng. Societies, Vol. XXI. 1898, p. 219.

Canopy Club, Inc., of Boston, a Masonic fraternity including all State employees; of the Boston Society of Civil Engineers; and of the Massachusetts Highway Association. From December 13, 1893, to December 31, 1918, he was a member of the New England Water Works Association. He was also a member of the Appalachian Mountain Club from 1898 to 1914, and at one time a member of the Massachusetts Bicycle Club. At the time of his death Mr. Wood was a member of the Alumni Council of the Massachusetts Institure of Technology, representing the Class of 1876.

On March 21, 1883, he was married to Emily Wyman Eaton, who survives him. He is also survived by his sister, Annie Louise Wood, and two of his three children, Dana M. Wood, M. Am. Soc. C. E., and Mildred Louise Wood.

Mr. Wood was imbued with a quiet efficiency. He was a lover of his fellow men, and always carried with him a cheerful optimism that was a help and a joy to his friends. Nothing delighted him more than to hear a good anecdote and to relate one. He will be missed by all who knew him.

Mr. Wood was elected a Member of the American Society of Civil Engineers on April 3, 1895. He had also been a member of the Northeastern Section of the Society since its organization.